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Page 1937, lines 10 and 13, for "Ocean Grove" read "Neptune Township".

Page 1938, Table 14, for "Ocean Grove" read "Neptune Township".

Page 1940, lines 21 and 24, for "Ocean Grove" read "Neptune Township".

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Page 623, line 8, for "Fig. 13" read "Fig. 7".

Page 984, line 18, for "Delaware" read "Chesapeake".

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1281

COLORADO RIVER SIPHON.*

By George Schobinger, Jun. Am. Soc. C. E.†

With Discussion by Messrs. H. T. Cory and George Schobinger.

Yuma is on the Arizona side of the Colorado River, a short distance below the mouth of the Gila River and about 25 miles north of the Mexican boundary. The bottom lands on both sides of these two rivers, leveed for protection against overflow, together with some mesa land, form the irrigable land of the Yuma Project of the United States Reclamation Service. The water for irrigation is taken from the Colorado at Laguna Dam, a diversion weir 14 miles above Yuma. To conduct the water to the Yuma Valley and Mesa, which is that portion of the Project on the Arizona side below Yuma, it was necessary to construct main canals to, and provide means of crossing, either the Colorado or the Gila. Both rivers have unstable beds, and are subject to sudden floods, which scour out deep channels. Owing to the relative elevations of the water in the canals and rivers, and the great variation between flood stage and low-water stage, a crossing by an aqueduct over either river would have been impossible, and an inverted siphon crossing was the only practicable one. For reasons of economy in the construction of both canal and crossing, the Colorado River siphon was selected. As may be seen on the general map, Fig. 1, the siphon is near Yuma, and a short distance below the Southern Pacific Railroad Bridge. The river at this point flows be-

*Presented at the meeting of May 7th, 1913
†Now Assoc. M. Am. Soc. C. E.

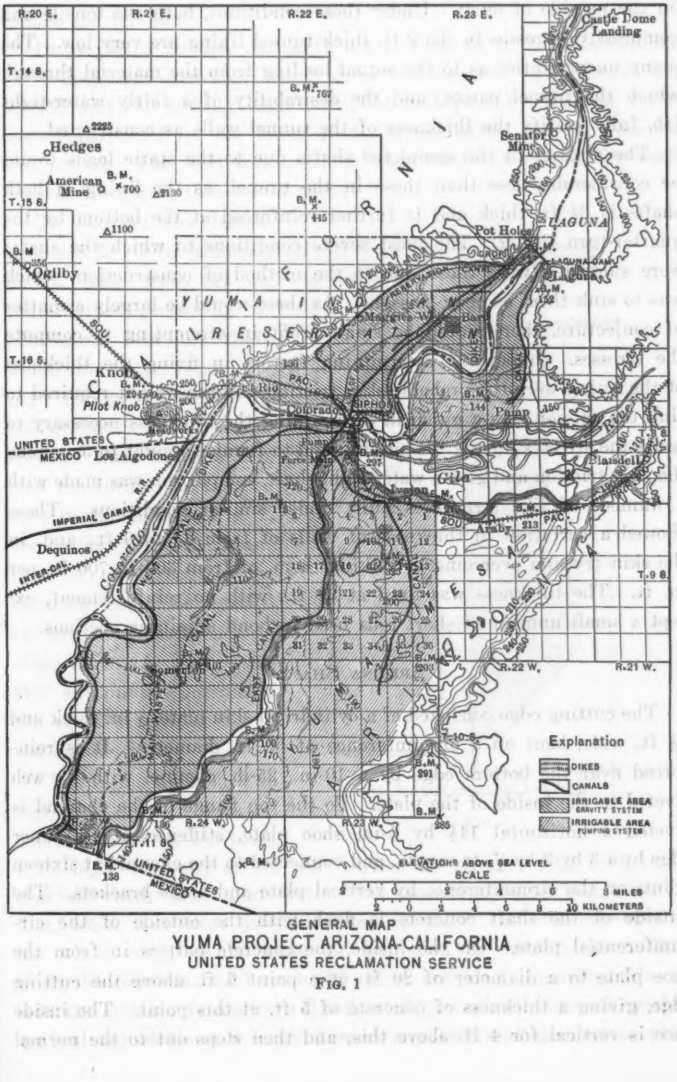
tween high banks which restrict its meandering tendencies and permit a shorter tunnel than would be possible at other points.

The variation between extreme flood and low-water surface elevations has been as great as 18 ft. This, however, does not represent the variation in cross-section; it has been said that for every foot the river rises it scours out its bed 2 ft., and the cross-sections on Fig. 9 illustrate the approximate truth of the statement. The deepest scour within recent years is shown; this section was taken at the river gauging station 600 ft. below the line of the tunnel, the width of the river being the same at both points. At the periodic high-flood years the scour is always approximately the same. In the intermediate low-flood years the 30-ft. silt blanket dropped by the falling river of the preceding years is only partly removed.

Borings taken on the line of the tunnel indicated a bed of soft sandstone underlying the silt. It was thought that, by working at a sufficient depth, open tunneling could be carried on without great interference from seepage water. Therefore, preparations were made for sinking shafts on each side of the river, with the intention of following this method of construction.

Before describing the construction in detail, a few words may be said of the general dimensions of the work. The California shaft is 130 ft. from the center of the levee, and 250 ft. from the Southern Pacific Railroad. The Arizona shaft is in high ground, 200 ft. from the river, and 955 ft. from the California shaft. The canals conducting the water to and from the shafts have a bottom width of 80 ft., a water depth of 7 ft., and carry 1 400 sec-ft., at full capacity, which is sufficient for the irrigation of about 100 000 acres. The siphon was designed to operate at full capacity with a loss of head of 2 ft. When operating at less than full capacity the same loss of head is obtained by throttling the flow at the intake; thus the water surface at the intake and outlet are maintained uniformly at Elevations 132 and 130, respectively. The inside diameter of the California shaft is 17 ft., and that of the Arizona shaft is 23 ft. The elevation of the tunnel flow line is + 49 at the shafts and + 47 in the middle of the river.

The assumed maximum conditions of loading to which the tunnel is subjected are, (*a*) tunnel empty, with pressure on the outside only, due to a hydrostatic head of 74 ft., at high stage of the river, and (*b*)



GENERAL MAP
YUMA PROJECT ARIZONA-CALIFORNIA
UNITED STATES RECLAMATION SERVICE

FIG. 1

system in operation, at low stage of the river, with pressure on the inside due to a hydrostatic head of 76 ft., partly balanced by a head on the outside of 59 ft. Under these conditions, both the tensile and compressive stresses in the 2-ft. thick tunnel lining are very low. The many uncertainties as to the actual loading from the material through which the tunnel passes, and the desirability of a fairly water-tight job, fully justify the thickness of the tunnel walls as constructed.

The stresses in the completed shafts due to the static loads would be considerably less than those in the tunnel, as the lining of both shafts is 3½ ft. thick and is further reinforced at the bottom by the quarter-turn lining. The most severe conditions to which the shafts were subjected were those due to the method of construction, which was to sink them as open caissons. As these would be largely a matter of conjecture, there would be small profit in attempting to compute the stresses. Important determining factors in fixing the thickness of the lining were the probable skin friction and the weight required to sink the caisson. As a guide in determining the thickness necessary to resist the stresses during the sinking and to furnish weight to overcome the skin friction and give a water-tight shaft, comparison was made with a number of like structures built under similar conditions. These showed a variation in thickness of walls of from 2½ to 6 ft., and, in the skin friction overcome in sinking them, of from 300 to 700 lb. per sq. ft. The thickness was fixed at 3½ ft., with no reinforcement, except a small number of short rods used to bond adjoining sections.

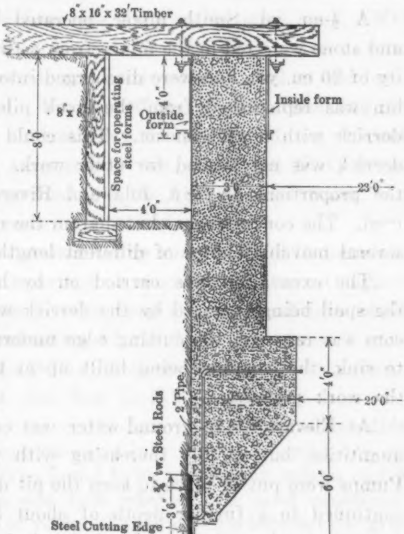
Arizona Shaft.

The cutting edge consisted of a cylindrical skin plate, ½ in. thick and 3½ ft. wide, bent on a circumference of 30 ft. diameter. It is reinforced near the bottom edge by a 10-in., 25-lb. channel with its web riveted to the inside of the plate. To the top flange of the channel is riveted a horizontal 14½ by ½-in. shoe plate, stiffened at its inner edge by a 3 by 3 by $\frac{7}{16}$-in. angle, and connected to the channel at sixteen points on the circumference by vertical plate and angle brackets. The outside of the shaft concrete is flush with the outside of the circumferential plate. On the inside, the concrete batters in from the shoe plate to a diameter of 20 ft. at a point 6 ft. above the cutting edge, giving a thickness of concrete of 5 ft. at this point. The inside face is vertical for 4 ft. above this, and then steps out to the normal

inside diameter of the shaft, which is 23 ft. The rivet heads on the outside of the cutting edge skin plate were countersunk, in order to reduce the friction.

In preparing to sink the shaft, a pit, some 40 ft. in diameter and 12 ft. deep, was excavated, using teams and scrapers. The bottom was leveled and the cutting edge assembled and placed. Wooden forms were constructed for the lower 10 ft. of the shaft, and the concrete was placed. The steel forms, which were used throughout the shaft construction, were then set in place.

To support the forms, twelve 8 by 16-in. timbers, 32 ft. long, were set on radial lines projecting as cantilevers over the shaft walls, the outer ends being firmly anchored in the ground. Bolts passing through these timbers carried the forms. As may be seen on the section, Fig. 2, the inside of the shaft was thus left open, so that excavation could be carried on without interference by other work. A passage was also left on the outside of the shaft to give access to the outside form.



HALF SECTIONAL ELEVATION

FIG. 2.

The forms were built in segments of a cylindrical surface, with $\frac{3}{16}$-in. plate, 4 ft. high, stiffened at top and bottom by circular ribs of built-up channels with horizontal web. The segments were connected so that, when they were assembled and in place, the circumference of the whole could be increased or decreased by turnbuckles. Thus, when the concrete had been built up, as previously described, the inside and outside forms were fitted to the completed concrete, with a vertical lap of about 2 in., and were plumbed, and clamped in place

by tightening the turnbuckles. No braces, ties, spreaders, or other means of support were required. When the forms had been filled with concrete, the hanger-bolts were released; excavation proceeded through the open space in the center of the shaft, and, as the caisson sank, the forms traveled with it. In preparing for a new lift of concrete, the turnbuckles were released, the forms were pried from the face of the concrete, raised to their original level with the derrick or tackles supported from the cantilever timbers, and reset as before. In this way a shaft wall of uniform thickness and very smooth surface was obtained.

A ½-cu. yd. Smith mixer, operated by steam, was used. Sand and stone were fed to the mixer from a two-pocket bin, having a capacity of 20 cu. yd., and were discharged into the measuring hopper. The bin was replenished from the stock piles directly back of it, by a derrick with a grab-bucket. This could be done at times when the derrick was not needed for other work. The concrete was mixed in the proportions 1 : 2½ : 5, Iola and Riverside Portland cement being used. The concrete was chuted from the mixer directly into the forms, several movable chutes of different lengths being used.

The excavation was carried on by hand, with pick and shovel, the spoil being removed by the derrick with a 1-cu. yd. bucket. The core was removed, the cutting edge undermined, and the shaft allowed to sink, the concrete being built up at the surface in 3½-ft. lifts as the work progressed.

At Elevation 120 ground-water was encountered, at first in small quantities, but rapidly increasing with the depth below this plane. Pumps were put in place to keep the pit dry, and hand excavation was continued to a further depth of about 50 ft. Here the excess hydrostatic head on the outside of the shaft was sufficient to cause sudden inrushes of water and sand under the cutting edge, and it was found necessary to change the method of excavation. The core was now removed by clam-shells to a depth of from 5 to 10 ft. below the cutting edge. Divers then drilled holes in the supporting ring under the cutting edge and placed small charges of dynamite. These, when fired, caused the shaft to sink varying distances.

When the cutting edge had penetrated some distance into the bed of soft sandstone indicated by the borings, attempts were made to seal the shaft to the undisturbed hard material by sinking pipes from

the surface outside of the caisson walls and forcing grout through them into the running sand. It was thought that by impregnating with grout the soft ground surrounding and for some distance above the tunnel opening, the flow of water might be sufficiently cut off to permit of open tunneling into unfissured, less pervious ground. After the grouting was complete a concrete plug was placed in the bottom of the shaft under water, and the water was pumped out. Preparations were made for tunneling operations. Cage guides and a gallows frame were constructed and hoisting machinery was erected. An open crib was built on top of the shaft, and a working floor laid. Tracks were laid from the mixer to the shaft and from the shaft to the spoil bank. While work was under way cutting through the shaft walls for a tunnel opening, other "blows" of water and sand occurred, and at the last of these the work was flooded. Further efforts at open tunneling were then abandoned, it being evident that the grout had not efficiently consolidated the material surrounding the shaft, and that the soft and porous nature of the ground and the high head of water would make it necessary to work under compressed air.

The foregoing method of consolidating running sand in foundation and similar work has been described as successful in several engineering writings. The outcome in this case showed conditions similar to those found in the construction of the New York tunnels of the Pennsylvania Railroad. The following is quoted from a discussion by the late C. L. Harrison, M. Am. Soc. C. E.*

"Grout was used extensively in the face in an effort to consolidate the material. In digging out the sand into which the grout had been injected under pressure, it was found to be collected in masses and not generally distributed; this was true whether the sand contained a large or a small percentage of voids. The interstices between the grains of sand were too small to permit the free flow of grout into them; however, grouting in the face did compact the material to some extent and reduce the flow of air through it."

Francis L. Sellew, M. Am. Soc. C. E., says:†

"In the writer's judgment, it is impracticable to so consolidate fine sand by the injection of grout that the resulting material will serve any useful purpose. While sufficient pressure may be used to cause penetration, the fluid will move at such low velocities that the grout will set up before any appreciable distance has been traveled.

* *Transactions*, Am. Soc. C. E., Vol. LXIX, p. 400.

† "The Colorado River Siphon," *Engineering News*, August 29th, 1912.

It is believed that success attained by this method will be confined to coarse materials."

CALIFORNIA SHAFT.

The work at the California shaft will not be described in detail, as the methods and results were similar to those at the Arizona shaft, except as to the dimensions of the shaft, which have been mentioned elsewhere. The ground through which the shaft passes is somewhat more varied in character, as may be seen by reference to Fig. 14. The final grade, as originally contemplated, was somewhat lower than on the Arizona side, but the difficulties of construction were such that the final elevation of the cutting edge was fixed at 28 ±; and the bottom of the shaft was sealed at that elevation.

WORK UNDER COMPRESSED AIR.

When it became evident that, owing to the shattered and porous condition of the material surrounding the shaft, open tunnel methods were out of the question, a Consulting Board, consisting of Louis C. Hill, M. Am. Soc. C. E., Supervising Engineer; Francis L. Sellew, M. Am. Soc. C. E., Project Engineer, of the United States Reclamation Service; and Silas H. Woodard, M. Am. Soc. C. E., formerly Division Engineer on the East River Tunnels of the Pennsylvania Railroad, made an investigation of the existing conditions, and advised a change in method to tunneling by the pneumatic process, the work to be carried on from the Arizona shaft only.

A compressor plant was rented from Charles A. Haskin and Company, Tunnel Contractors. Owing to the impossibility of securing suitable labor, skilled in such work, in that part of the country, experienced "sand-hogs" were brought from the East, to form a nucleus of trained compressed-air tunnel men. E. C. Hayden, Assoc. M. Am. Soc. C. E., was Superintendent of Construction on the work from this time until the successful completion of the tunnel. The rented equipment was as follows: Three Ingersoll compressors: 20 by 30-in., 24½ by 24-in., and 17 by 24-in; vertical and horizontal air locks and equipment, medical lock, air receivers and coolers, and the 4-ft. steel shafting used above the air deck.

The rated capacity of the compressor plant was 4 000 cu. ft. of free air per minute compressed to 40 lb. The maximum consumption for 24 hours averaged 2 000 cu. ft. per min., or about 55% of the rated capacity. The maximum recorded consumption for a short time

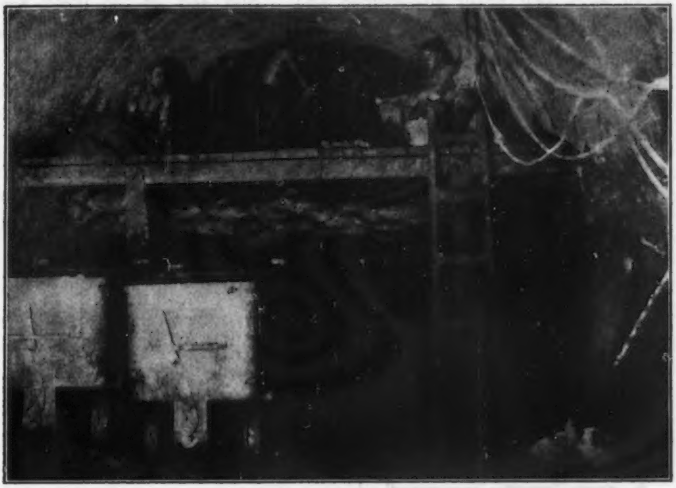


FIG. 3.—SAND-BAG BULKHEAD. EXCAVATING IN HEADING.

FIG. 4.—VIEW OF TUNNEL.

was 2 200 cu. ft. per min. The compressor plant was thus never seriously taxed for a long period. The average consumption during the tunneling operations was 1 450 cu. ft. of free air per minute, to a pressure of 28 lb. per sq. in.

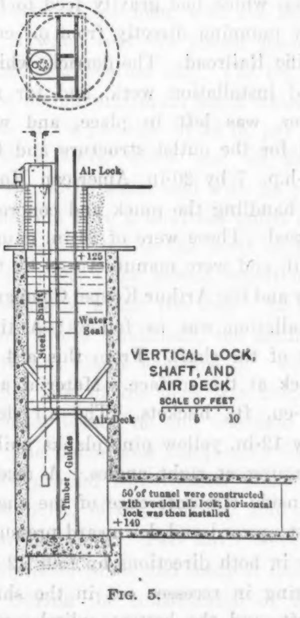


FIG. 5.

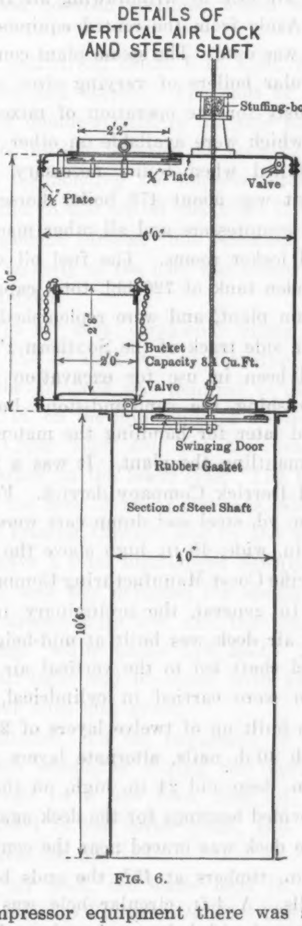


FIG. 6.

In addition to the foregoing compressor equipment there was already installed a 12½ by 14-in. Ingersoll-Rand high-pressure machine, with a capacity of 200 cu. ft. per min. at 100 lb. This had been used during grouting operations at the Arizona shaft, and, during the

further prosecution of the work, was used for operating drills and the concrete lock, and also as emergency equipment at times when the consumption of air was great, or when it was not advisable to fill the air lock by withdrawing air from the heading.

Aside from the rented equipment, that which was already on the job was used. The steam plant consisted of five oil-burning, horizontal, tubular boilers of varying sizes, some of which had been used previously for the operation of mixers, hoists, derricks, etc., and others of which were available on other parts of the Project and were later installed when found necessary. The total rated capacity of the plant was about 475 boiler horse-power. This served for operating the compressors and all other machinery, and for heating the change and locker rooms. The fuel oil was stored in one concrete and one wooden tank of 720 bbl. total capacity, which had gravity feed to the steam plant, and were replenished by pumping directly from oil cars on a side track of the Southern Pacific Railroad. The derrick, which had been in use for excavation and installation work, and for replenishing the sand-and-stone hopper, was left in place, and was used later for handling the material for the outlet structure and for dismantling the plant. It was a 20-h.p., 7 by 20-in. American Hoist and Derrick Company derrick. For handling the muck and concrete, ½-cu. yd. steel end dump cars were used. These were of 24-in. gauge, 24 in. wide, 40 in. high above the rail, and were manufactured by the Pacific Coast Manufacturing Company and the Arthur Koppel Company.

In general, the preliminary installation was as follows: A timber air deck was built at mid-height of the shaft. From this a 4-ft. steel shaft led to the vertical air lock at the surface. Material and men were carried in cylindrical, 8-cu. ft., buckets. The air deck was built up of twelve layers of 2 by 12-in. yellow pine planks nailed with 40-d. nails, alternate layers running at right angles. A recess, 4 in. deep and 24 in. high, on the inside circumference of the shaft, provided bearings for the deck against upward and downward pressure. The deck was braced near the center in both directions by four 12 by 12-in. timbers at 45°, the ends bearing in recesses cut in the shaft walls. A 4-ft. circular hole was left, and the bottom cylinder was fastened with bolts passing through its flanges and the air deck. Other holes were left for pipes for air supply, electric light, and telephone wires.

From the deck to the surface was 72 ft. Six sections of 4-ft. cylindrical steel shaft were used. These were made up of ¼-in. steel plate with lap-riveted joints and 4 by 4 by ½-in. angle flanges at the top and bottom. Gaskets were used between sections, which were bolted together through the flanges.

The vertical air lock was a 6-ft. cylindrical drum, 6 ft. high, with 2½-ft. circular plate doors in the top and bottom. The ends of the drum were of ⅝-in. plate, dished and double-riveted to the side plate, which was ⅜ in. thick. The 8-cu. ft. buckets cleared the opening by about 1 in. all around. The cable for hoisting the buckets through the steel shaft passed through a gland in the top of the lock; it was of especially compact texture and smooth surface, thus reducing the leakage of air to a minimum. The capacity of the lock was about five buckets, or ten men, with the lock-tender.

A water seal was placed above the air deck, completely filling the shaft to Elevation 129. The top of the air deck was at Elevation 85. The actual maximum loads which the air deck was required to carry were (*a*) a pressure of 36 lb. per sq. in. on the lower side of the deck, with a 43-ft. water seal on the upper side, and (*b*) the weight of the 43-ft. water seal with no pressure on the lower side. The first condition gave a total net uplift on the deck of about 500 tons, which was transmitted to the shaft by the bracing timbers, and by the deck at its edge. The second condition gave a total unbalanced downward pressure of about 700 tons. As the deck was designed for somewhat more severe conditions, the actual stresses were low, and the timber gave but slight evidence of "working" while under full load. The under side of the deck was made air-tight and fire-proof with sheet asbestos tacked on and plastered over with clay-grout.

The boiler and compressor installation and the remainder of the above-ground construction plant were completed during this period. The hoist, mixer, derrick, and change room had been used on the previous work. Two units were added to the boiler plant, and the compressor plant and medical lock were set up.

During the construction of the air deck, the water level in the shaft had been kept down by pumps about to Elevation + 80. The air pressure was now raised and the water blown and pumped out. The relation between the river water surface and that in the shaft, with the air pressure, showed that the soft and porous material surrounding

and underlying the shaft offered small resistance to the transmission of hydrostatic pressure. When the water level had been lowered to + 42, the pressure was 36 lb. per sq. in. At that time the river water surface was at about 122. Elevation 42 was set as the bottom elevation of the concrete plug. Allowing for the thickness of the plug, and the sump, this brought the elevation of the flow line of the tunnel at + 49. The following considerations influenced the fixing of this grade:

First, the tunnel constructed from this elevation would have a minimum cover at high stage of the river of about 15 ft., which was considered sufficient for safety of construction in this material. As a matter of history, the tunnel did not reach the deepest point of the river until after the flood period, and the cover at that time was more than 30 ft.

Second, there was no assurance that the ground would be more favorable lower down.

Third, the difficulties, slowness, and expense of construction increase greatly with higher pressures.

The plug in the bottom of the shaft was designed as a circular reinforced concrete diaphragm, supported at the edge, to resist the maximum hydrostatic pressure, which at highest stage of the river would be about 40 lb. per sq. in. It seemed probable that the diaphragm would receive its full load while the concrete was still comparatively new. In order to make it possible to relieve the pressure somewhat, ten 2-in. pipes were set, to pass through the plug; the lower ends were nested in crushed rock to insure a free opening, and the upper ends were furnished with globe valves.

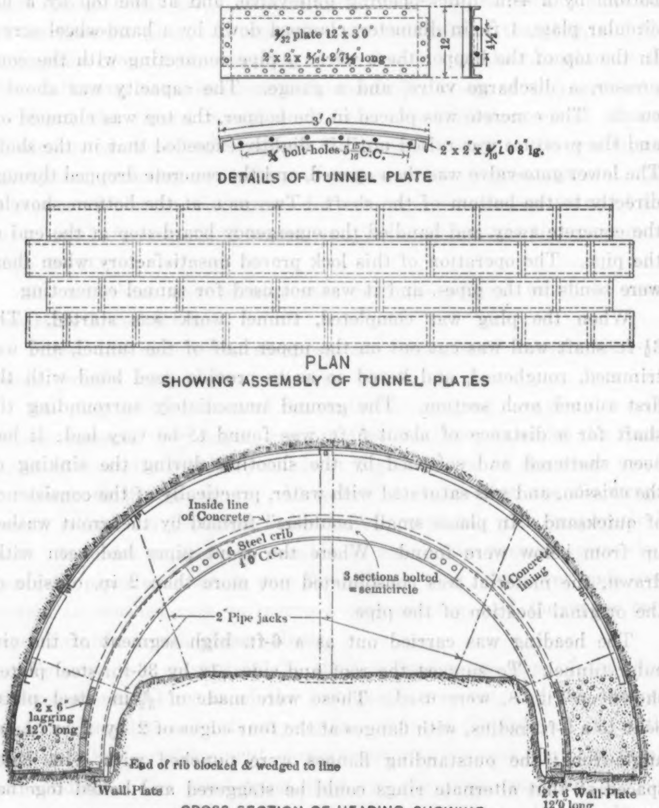
To provide a support in the shaft walls for the edge of the plug, a circumferential recess of the full height was cut, and the concrete was roughened to a depth varying from 3 in. at the top to 7 in. at the bottom. Any reinforcement encountered was cut and bent out to aid in bonding. The reinforcing steel for the plug consisted of $\frac{3}{4}$-in. and 1-in. square twisted rods, and old 15-lb. industrial rail. It was lowered into the shaft through an improvised pipe-lock. A 4-in. pipe was used, with a gate-valve at the top and a board flap-valve at the bottom. The bottom being capped, the steel was lowered from the top until its end rested on the board-valve. The top was then closed and the pressure equalized; the rods were dropped through and on a platform, and were then set in place. In this way steel was lowered as

fast as it could be set. Concrete was lowered in buckets through the 4-ft. steel shaft from the vertical air lock at the surface, and also through a small material lock. This consisted of a hopper closed at the bottom by a 4-in. quick-opening gate-valve, and at the top by a flat circular plate, 1 ft. in diameter, clamped down by a hand-wheel screw. In the top of the hopper there were: a pipe connecting with the compressor, a discharge valve, and a gauge. The capacity was about 3 cu. ft. The concrete was placed in the hopper the top was clamped on, and the pressure was raised until it slightly exceeded that in the shaft. The lower gate-valve was then opened, and the concrete dropped through directly to the bottom of the shaft. Two men at the bottom shoveled the concrete away and handled the emergency board-stop at the end of the pipe. The operation of this lock proved unsatisfactory when there were bends in the pipes, and it was not used for tunnel concreting.

When the plug was completed, tunnel work was started. The $3\frac{1}{2}$-ft. shaft wall was cut out on the upper half of the tunnel, and was trimmed, roughened, and keyed so as to provide good bond with the first tunnel arch section. The ground immediately surrounding the shaft for a distance of about 5 ft. was found to be very bad; it had been shattered and softened by the shooting during the sinking of the caisson, and was saturated with water, practically of the consistency of quicksand. In places small "boulders" formed by the grout washed up from below were found. Where the grout pipes had been withdrawn, the material was consolidated not more than 2 in. outside of the original location of the pipe.

The heading was carried out as a 6-ft. high segment of the circular tunnel. To support the roof and sides, 12 by 36-in. steel plates, shown on Fig. 8, were used. These were made of $\frac{3}{32}$-in. steel plate, bent to a 9-ft. radius, with flanges at the four edges of 2 by 2 by $\frac{5}{16}$-in. angle-irons; the outstanding flanges were punched with $\frac{3}{4}$-in. holes spaced so that alternate rings could be staggered and bolted together with $\frac{5}{8}$-in. bolts. The breast was boarded and clayed tight. The ground was cut out on the circumference the size of one plate, and the plate set and bolted. The space between the plate and the roof was filled with clay. In this way a ring was completed from the crown to about 3 ft. above the springing line on each side. Trench braces were set radially on every second or third ring. The breast was then cut down level with the bench breast-boards, reset, and clayed, and the process

repeated. The joints between plates and between rings of plates were clayed over and mud-washed at intervals. Empty sacks and clay sacks were used for " blows."



CROSS-SECTION OF HEADING SHOWING TUNNEL PLATES, BRACING, CONCRETE FORMS, ETC.

FIG. 7.

When an advance of fourteen rings beyond the finished work had been made, the heading, roof, and breast were tightened, the pressure was raised from 2 to 3 lb., and side trenches were cut to the springing line. Wall-plates were set to line and grade, steel forms set, and the 2-ft. concrete lining was placed. The forms were 6-in. channels bent

to a radius of 6 ft. 10 in., the 6-in. dimension being radial; they were in three sections for the semicircle, and fastened with ⅝-in. bolts. The lagging was 2 by 6-in. surfaced lumber, 12 ft. long. The longitudinal joint at the springing line battered from face to back in such a way as to facilitate grouting and caulking the joint on the completion of the invert concrete.

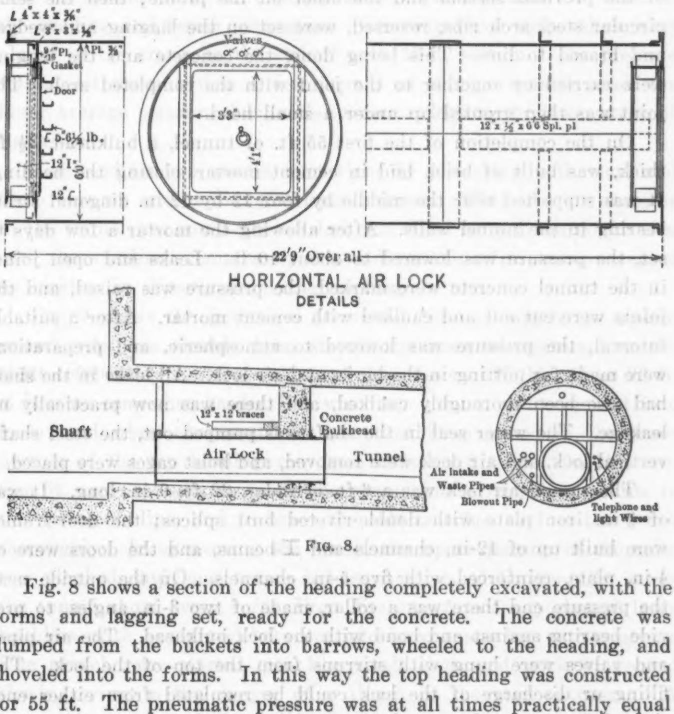


Fig. 8.

Fig. 8 shows a section of the heading completely excavated, with the forms and lagging set, ready for the concrete. The concrete was dumped from the buckets into barrows, wheeled to the heading, and shoveled into the forms. In this way the top heading was constructed for 55 ft. The pneumatic pressure was at all times practically equal to the hydrostatic head. The bench was then excavated in sections of from 10 to 12 ft. For this the pressure required was from 4 to 6 lb. higher than for the heading. Sections of bench lapped with sections of heading, so that there was no joint in the entire tunnel on one plane. The arch concrete carried itself on about a 10-ft. span longitudinally, during the concreting of a section of invert. In excavating the bottom, it was seldom necessary to brace the sides, except immediately

under the arch concrete. Here 16-ft. trench braces were set across the full width. The sand was kept plastered with clay to within a few feet of the bottom. When the excavation was completed, profile bulkheads were set at the end, to line and grade, and the bottom was filled with concrete to a level with the tops of the profiles. On the concrete there was then laid 12-ft. lagging, one end lying on the finished work of the previous section and the other on the profile; then the semicircular steel arch ribs, reversed, were set on the lagging and wedged and braced to line. This being done, the concrete and the lagging were carried up together to the joint with the completed arch. The joint was then grouted up under a small head.

On the completion of the first 55 ft. of tunnel, a bulkhead, 3½ ft. thick, was built of brick laid in cement mortar, closing the heading. It was supported near the middle by four 12 by 12-in. diagonal struts bearing in the tunnel walls. After allowing the mortar a few days to set, the pressure was lowered to about 10 lb. Leaks and open joints in the tunnel concrete were marked, the pressure was raised, and the joints were cut out and caulked with cement mortar. After a suitable interval, the pressure was lowered to atmospheric, and preparations were made for putting in the horizontal air lock. All leaks in the shaft had also been thoroughly caulked, and there was now practically no leakage. The water seal in the shaft was pumped out, the steel shaft, vertical lock, and air deck were removed, and hoist cages were placed.

The tunnel air lock was a 6-ft. cylinder, 22 ft. 9 in. long. It was of ⅜-in. iron plate with double-riveted butt splices; the door-frames were built up of 12-in. channels and I-beams, and the doors were of ½-in. plate, reinforced with five 5-in. channels. On the outside near the pressure end there was a collar, made of two 3-in. angles, to provide bearing against and bond with the lock bulkhead. The air pipes and valves were hung with stirrups from the top of the lock. The filling or discharge of the lock could be regulated from either end, there being a double set of 2-in. supply and discharge valves. The capacity of the lock was five ½-cu. yd. cars.

The bulkhead was of plain 1 : 2½ : 5 concrete, 4 ft. thick, with two sets of diagonal braces against the tunnel walls. Bearing on the circumference was provided by a 3-in. recess which had been left in the concrete lining on this section. Pipes for air supply, telephone and electric light wires, and drainage were concreted in.

When this work had been completed (October 20th, 1911), the pressure was again raised and the temporary brick bulkhead was cut out. The method of advancing the heading has already been outlined, and in general this method was adhered to throughout the construction. Sandbag and clay bulkheads were placed in the lower half of the tunnel at the end of the completed work during the advance of the heading, the pressure usually varying from 3 to 5 lb. on the heading and bench excavation. Grouting pipes with valves were placed in the crown at the end of each 12-ft. section. The pressure was at all times within a few pounds of the hydrostatic head, the lower average pressure during the winter reflecting the lower stage of the river.

The heading was always kept tight by plastering with clay, and there were no serious blows of long duration. While the air chamber in the tunnel was small, the air lock was filled by a direct connection with the auxiliary compressor, to avoid variations in pressure caused by locking in directly from the heading.

Fig. 9 shows the geological formation through which the siphon passed. As has already been stated, the material surrounding the shaft softened during the sinking of the caisson. Within 15 or 20 ft. it became somewhat harder, but was very porous, and crumbled into loose sand when excavated. A pocket of loose gravel in the lower portion of the top heading ran from Stations 3 to 5. On this section it was necessary to carry the tunnel plates or poling boards to the springing line, and to brace the side trenches against the bench thoroughly. In excavating the bench, the upper portion of the sides was planked, and long trench braces were placed horizontally across the tunnel about every 4 ft. From Stations 5 to 7 a pocket of large boulders, with pea gravel between, interfered somewhat with progress. The gravel, once started, ran like small shot, and work progressed slowly, owing to the danger of a slump in the roof or breast. A lense of exceedingly hard and thoroughly cemented gravel, or puddingstone, was encountered between Stations 5 and 6. The last 300 ft. brought to light various pockets, large and small, of boulders, clay, gravel, and quicksand. Both experienced prospectors and amateur miners were disappointed in not finding gold-bearing gravel under the Colorado, although an incipient boom was started by salting a car of muck with brass filings.

From Station 7 to the California shaft the material was somewhat more favorable, and progress was steady. On April 12th a galvanized-iron well casing was encountered. This was one of the original test

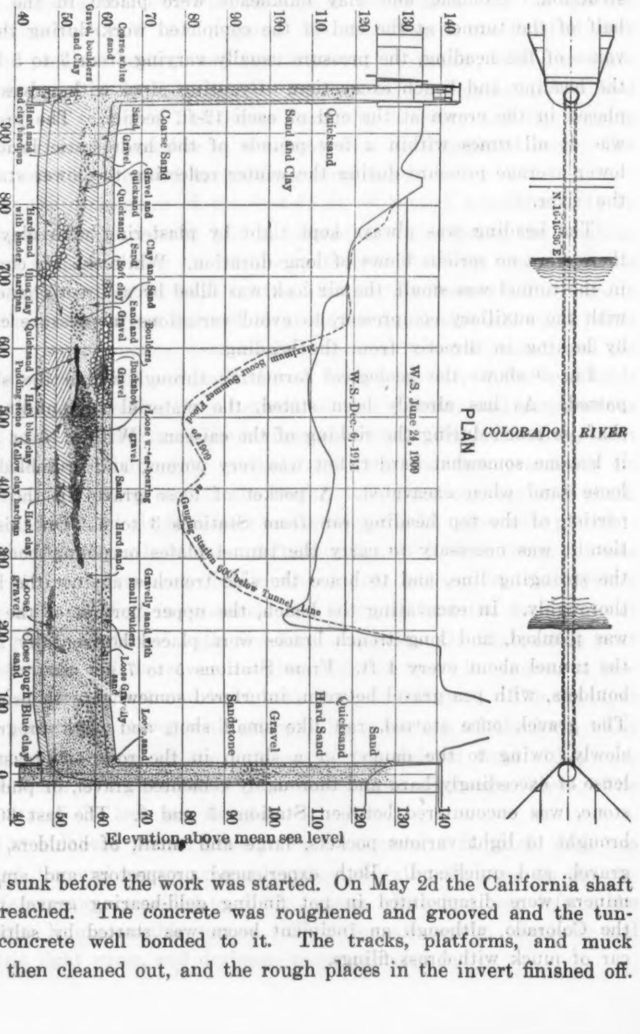


Fig. 9.

wells sunk before the work was started. On May 2d the California shaft was reached. The concrete was roughened and grooved and the tunnel concrete well bonded to it. The tracks, platforms, and muck were then cleaned out, and the rough places in the invert finished off.

The pressure was dropped to 5 lb., and all leaks, cracks, and porous spots were marked; then the pressure was raised to 30 lb., and the leaks were caulked. The tunnel was then opened for inspection. The leaks remaining were insignificant, and the work all appeared to be in good condition. The lock bulkhead, lock, and cage guides were removed, and the quarter turn in the bottom of the Arizona shaft, described elsewhere, was constructed.

From the time of the installation of the horizontal lock to the time of completion of the tunnel, the average progress was 4½ ft. per day. The best monthly progress was 165 ft., in favorable ground, during January, 1912. The slowest progress was during February, in gravel, boulders, and sand; in this month a length of 100 ft. of tunnel was completed.

Alignment.

The elevation of the flow-line at the shafts is + 49. For 100 ft. from each shaft the grade is 2% down, and between these points the bottom is level. The line is straight between shafts.

On the surface a parallel offset line was run, there being obstructions on the direct line. After pumping out the Arizona shaft, and previous to the construction of the air deck, the surface line was plumbed down and plugs were set in the shaft walls. After the installation of the horizontal air lock, the surface line was brought down and carried through the lock four times, to eliminate errors. The detailed operation was as follows:

The transitman, setting up on the surface line, directed the placing of two 2 by 3-in. wrought-iron staples, on each side of the shaft, the line falling within the cross-bar of each staple. These were driven into the solid crib timbers inclined over the shaft, at such an angle that the plumb line, hanging over them, would clear all obstructions below. The line was marked on the bar with a file, and the steel wire plumb lines, carrying 20-lb. plumb-bobs, were placed in the file marks. The transitman, by moving the telescope only slightly, or not at all, in a vertical direction, could then observe both wires directly below the points of support, sighting between the legs and over the top of the nearer staple. The plumb-bobs were hung in pails of water. The length of the base line thus obtained was 10 ft. 6 in.

A 4 by 6-in. timber was placed across the tunnel end of the air lock, and securely wedged and braced. The transit, on a small trivet,

was set on this timber and lined in by sighting on the plumb wires, oiled paper and a candle being used for illumination. A scratch was made on line on the far end of the lock, above the door. A second transitman stayed in the shaft to observe the lock for motion due to jarring or working while the pressure was raised. The first transitman then locked through, checked his setting on the mark over the lock door, plunged his transit and took readings on two scales fixed in the tunnel roof at 30 and 100 ft. from the lock. Without lifting the transit, the operator again locked out and checked on the position of the plumb wires in the shaft. If the check was unsatisfactory, the line was again locked through and back. The transit was once more set up on the surface and the position of the wires was checked back to the original line, thus completing the circuit. While the alignment work was being done, one of the two cages was out of commission, the plumb lines occupying the one cage well. The work was done at a time when all material could easily be handled with one cage.

The level-work differed in no way from surface leveling, a point in the lock being used as a turning point. At each setting of the forms, a grade nail was set in the breast.

In carrying forward the line as the work in the tunnel progressed, two 2-in. wrought-iron staples were driven into the roof at the end of every 60-ft. section, on line and about 15 ft. apart. The exact line was marked with a file on the cross-bar of each staple, and a plumb line was hung in the file mark. The work of setting the forms preceding each run of concrete was done with a rule, a string, a plumb-bob, and a carpenter's level.

The string was stretched to the breast, tangent to plumb-lines hung in the file marks, and nails were set in the breast and at the end of the completed work. Between these was stretched a line, from which the foreman afterward could check the setting. Measuring from this line, the wall-plates were set in the side trenches, exactly to line, and approximately to grade. The first form was brought in, bolted up, and set nearest the breast, the ends, which were on the horizontal diameter, being braced securely to line, as given by the wall-plates on which they rested. The grade was then checked from the nail in the breast, and the rib was shimmed up to the proper level. The intermediate ribs were then set up and quickly aligned between the pilot form and the completed concrete. The cross-section was then measured

up, and what slight trimming and shaping remained was done in a short time. The concrete of the preceding section was cleaned and roughened, and the new run started.

The setting of the forms for the invert was somewhat simpler. A profile-bulkhead was constructed, made up of three thicknesses of 1-in. planks, the radii of the upper and lower edges of the profile corresponding to the inner and outer radii of the concrete lining. It was cut on radial lines into interchangeable sections 3 ft. long, the edges being bound with iron and carefully trued. When the excavation of a 10-ft. length of invert was complete, the profile was set up at the end of the section, the line and grade being measured from the completed concrete of the arch. The bulkhead was secured on the concrete face by iron pins driven into the ground, and was wedged and braced against the bench. As has been previously outlined, the bottom was filled with concrete; 12-ft. lagging was then laid on the concrete, one end resting on the completed work and the other end on the profile. The arch ribs, reversed, were set on the lagging, and secured against uplift by wedges and a cross-brace. The lagging and the concrete were then carried up together. The same set of profiles was used throughout the work. It was impossible to check the line and grade at the California shaft until the tunnel lining was complete and sealed to the shaft. The differences, however, were inappreciable, and the adjustment was made in the 3½-ft. thickness of the shaft lining.

Caisson Disease.

Doctors Henri ApJohn and O. I. Tower were medical examiners for the Reclamation Service. A certificate signed by the examiners was required before a man was employed at tunnel work.

The proportion of green men was necessarily very high, as there has been no compressed air work of any magnitude in the region supplying Yuma with labor. About twenty seasoned men, including the foremen, were brought from the East, and this group remained throughout the work. The remainder of the force shifted constantly; it was composed largely of Mexicans and "floaters" who came to spend the winter in Yuma. The conditions, therefore, were unfavorable, as far as the physical make-up and the seasoning of the laborers were concerned.

At the start, when the pressure rose above 30 lb., it was found

necessary to cut down the time spent under compressed air, so that the gangs worked a total of 4 hours in the heading during an 8-hour shift. The larger number of cases of "bends" at the start, under conditions apparently favorable as to air supply, was undoubtedly due to the fact that the highest pressures, averaging more than 35 lb., came at a time when the majority of the force was composed of green men. However, with the short shifts, there were no fatal cases. Later, although the pressure was at times as low as 27 lb., the same hours were adhered to, it being impracticable to vary the schedules with the varying pressures.

The time of locking out was fixed at from 12 to 15 min., the rate of decompression being made constant. This rule was rarely violated, except by the lock tenders, who were in and out at very short intervals, and whose tissues were never "saturated" by air under pressure. A medical lock, similar to those used on the East River Tunnels, was used for recompression and treatment of cases of "bends."

The total number of different men employed on compressed air work, from start to finish, was about 1 300. All these men had at one time to pass the medical examination. Some who were passed at first, but who were very susceptible, were eliminated later. However, in those cases in which the symptoms were merely pains in the extremities, which were relieved by recompression or local treatment, the men were at liberty to return to work at their pleasure. As the work was done in a small town, and the men lived at the Government bunk-house or at the other rooming houses close at hand, there were probably no serious cases which escaped the notice of the physicians. The records show the following cases treated:

(1) Cases causing disability for a few hours.......	299
(2) Cases causing disability from one to three days.	38
(3) Cases of partial paralysis, including bladder and lower extremities, causing disability for a few weeks	4
(4) Cases of total paralysis, resulting in death in three weeks..................................	1

Those in Groups (1) and (2) were largely cases of pains in the arms or legs, with a few cases of vertigo. These were numerous at the start; the most susceptible men, having suffered once, were usually

willing to leave the work after the experience; these cases, therefore, became of less account as the work progressed. The cases in Group (3) were not traceable to extraordinary conditions on the work at the time of their occurrence. They were scattered, and did not coincide with any noticeable increase in the other cases, and must be laid to unknown factors in the condition of the men at the time. The one death was of a seasoned man, who had passed the medical test and had worked for some time. His attack of paralysis followed a prolonged period of dissipation which left him in poor health and unable to withstand conditions which he might otherwise have weathered successfully. The record of only one death in the large number of men employed, and that not directly chargeable to conditions on the work, in a job which lasted for 10 months, is fairly satisfactory, as compared with other records.

The experience at the Colorado Siphon probably adds nothing new to what is known of the contributing causes and the treament of caisson disease. These have been fully discussed in recent papers presented to the Society or published in engineering journals.

California Shaft—Intake Structure.

The condition at the California shaft at the commencement of pneumatic work has already been described. The work remaining to be done at that side of the river was, first, the construction of a plug, at the same elevation as on the other side; second, arrangements for making the junction from the tunnel end and cutting through the shaft wall without loss of pressure; and, third, the construction of an intake structure.

The plug requires only brief mention. It was of reinforced concrete, 4 ft. thick, and 17 ft. in diameter; and was bonded and keyed to the shaft wall, as on the Arizona side.

Above this there was constructed a quarter turn in the shaft, to change the direction of flow of the water from vertical to horizontal. This was of concrete, and built on a center line radius of 8.5 ft. Above the quarter turn there was a reducer, 15 ft. long, from the shaft diameter of 17 ft. to the tunnel diameter of 14 ft.

The tunnel opening into the shaft was to be between Elevations 49 and 63. A vertical timber bulkhead, of two thicknesses of 12 by 12-in. timbers, bolted and keyed together, was built across the proposed

opening. The ends of the timbers were recessed into the concrete, and a flooring and roofing of 12 by 12-in. timbers placed. The space was filled with adobe. In this way, when the tunnel was cut through, it would open into a compartment entirely separated from the main shaft by a pressure-tight bulkhead, and a strong, water-tight joint could be made. The shaft outside of the bulkhead was filled with sand and water to Elevation 120.

During the construction of the tunnel the work on the intake structure was completed. This, together with the main check and wasteway, briefly described later, controls the flow of water through the upper portion of the main canal in Yuma Valley. As may be seen

BULKHEAD IN CALIFORNIA SHAFT

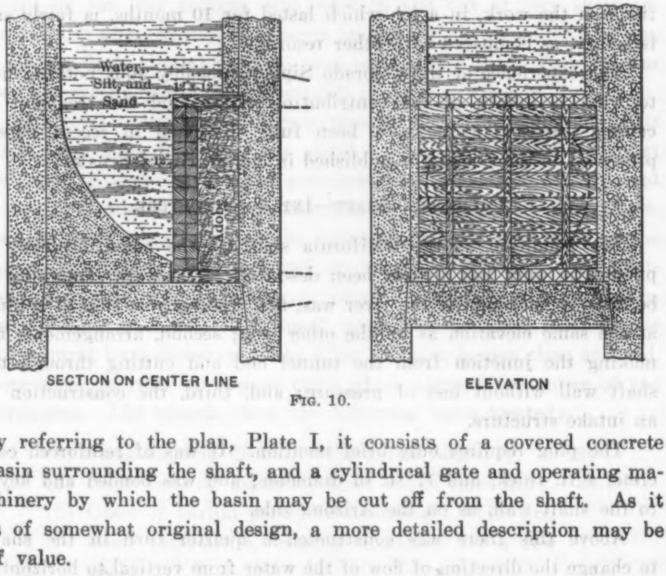
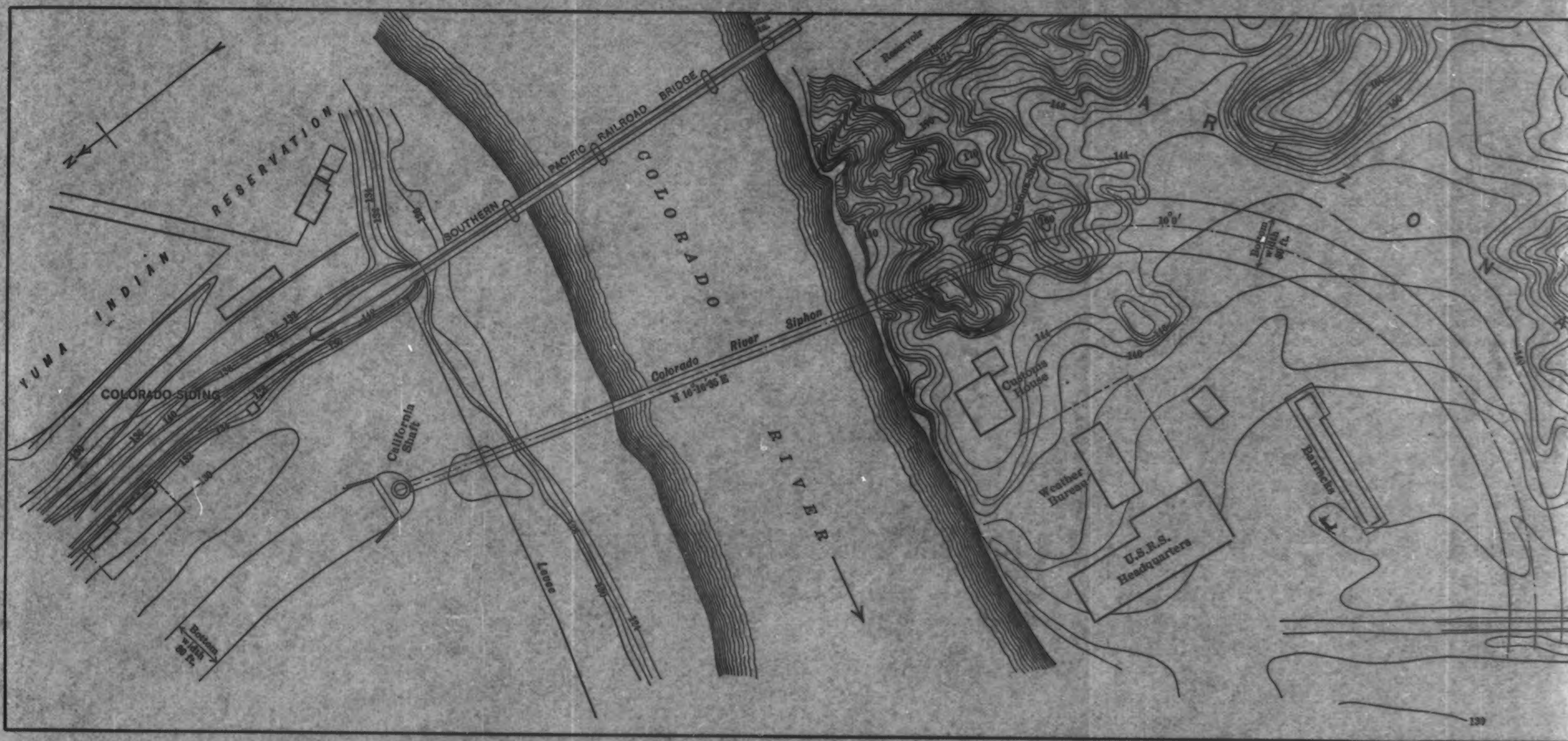


Fig. 10.

by referring to the plan, Plate I, it consists of a covered concrete basin surrounding the shaft, and a cylindrical gate and operating machinery by which the basin may be cut off from the shaft. As it is of somewhat original design, a more detailed description may be of value.

The main canal at this point is of trapezoidal section, having a bottom width of 80 ft. and a water depth of 7 ft. Warped retaining walls reduce this to a rectangular section 72 ft. wide and 7 ft. deep. From a line 15 ft. in front of the center line of the shaft, these walls, 11 ft. high, swing in, and around the shaft on curves of 36 ft. radius, compounded with curves of 17 ft. radius. At the rear of the shaft,



N
YUMA INDIAN RESERVATION
COLORADO SIDING
SOUTHERN PACIFIC RAILROAD BRIDGE
COLORADO RIVER
Colorado River Siphon
California Shaft
Levee
Reservoir
Custom House
Weather Bureau
U.S.R.S. Headquarters
Barracks

RAILROAD BRIDGE
Reservoir
COLORADO
RIVER
A
R
Z
O
Colorado River Siphon
N 16°16'35" E
10°0'
Bottom width 80 ft.
Customs House
Weather Bureau
U.S.R.S. Headquarters
Barracks
148
144
140

PLATE I.
TRANS. AM. SOC. CIV. ENGRS.
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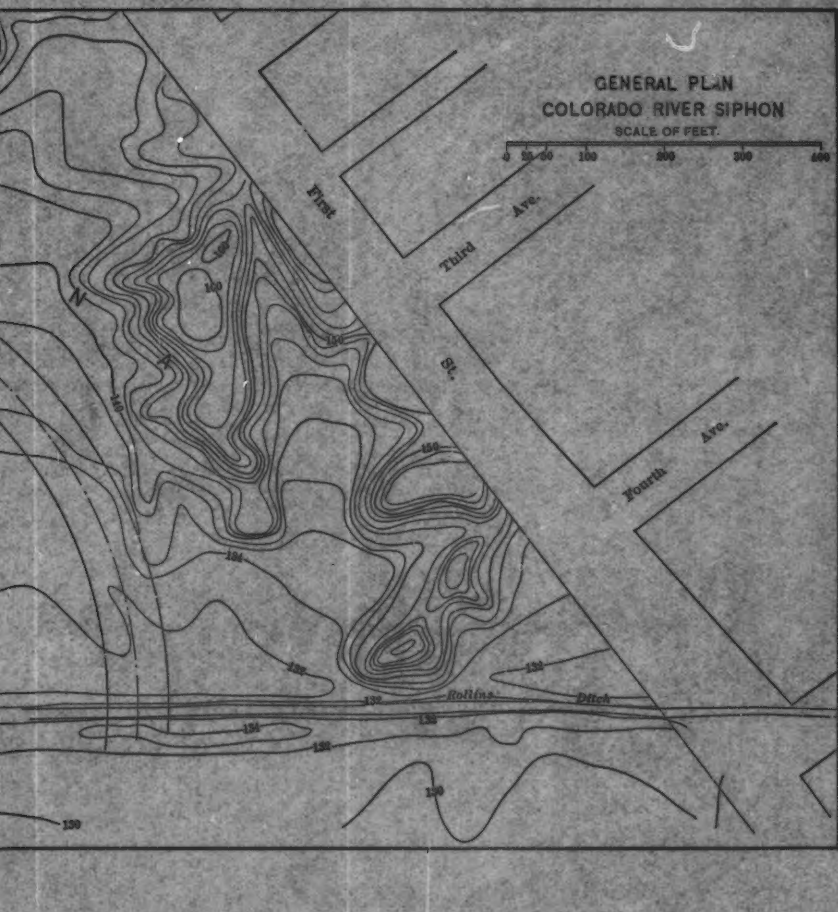
SCHOBINGER ON
COLORADO RIVER SIPHON.



where these walls meet, and at points 120° from it, are constructed three piers, enclosing the guides for the cylinder gate. The floor of the basin is 1 ft. thick, with the top at Elevation 125, which is also the elevation of the canal bottom. The flow of water is directly down into the shaft, there being no obstruction except the guide piers. The whole basin is roofed over with a reinforced concrete girder floor, which is on a level with the top of the canal banks. On this floor is built the operating tower, into which the gate is drawn when the siphon is in operation, and which carries the operating machinery.

The gate has been described as a "bottomless tin can", set over the top of the shaft; it is 8 ft. high and 21 ft. in diameter. When it is down on the sill, its top is 1 ft. above the normal water level in the canal. The skin plate is $\frac{3}{8}$ in. thick, stiffened at the top and bottom by rings of 3 by $3\frac{1}{2}$ by $\frac{3}{8}$-in. angles, which in turn are supported by 3 by $3\frac{1}{2}$ by $\frac{7}{16}$-in. angles as chords. The whole is held rigidly to a circular shape by three radial stiffening or tie-brace trusses, at 120° from each other, connected to a cast-steel, three-winged center post. The gate-stems are secured to the gate at these three points on the circumference, as are also the guide-bearing castings.

There are two types of guides, designated *A* and *B*. The two Type *A* guides merely present a free bearing surface, which constrains the gate only in a radial direction. Type *B* guide, which is embedded in the heavy rear pier, encloses a portion of the bearing casting on the gate, and thus prevents both radial and circumferential motion. All three guides are single castings, 19 ft. 5 in. long, the lower 2 ft. being embedded in the foundation. They are anchored in the concrete piers throughout their length, and are fixed at the top by a triangular lattice girder frame which is embedded within the girders of the operating-tower floor. The guides and bearings are of cast iron, with machined wearing faces.

The circular cast-iron sill on which the gate rests is shown in section with the gate on Plate II. It is made in six sections, bolted together, and anchored in the concrete. It is bolted to the guides at the three points of contact. In placing the sill, the shaft was trimmed out roughly to form a bed, and the sill was brought to grade with iron shims, and grouted in place.

As previously stated, the cover of the basin is a slab and girder floor. It is surrounded on all sides with a 2-in. iron-pipe railing. Above it is the operating tower, of cylindrical form, 8 ft. $8\frac{3}{4}$ in. high.

which carries the gate-stand and operating machinery, and also forms a sleeve into which the gate may be drawn. The three gate-stands, on two shafts at right angles to each other, are operated through a gear drive by a 3-h.p. continuous-current motor. The gate weighs about 8 tons, and in ordinary operation is raised and lowered at the rate of about 2 ft. per min. The tower is accessible from the floor of the structure by a concrete stairway. Four cast-iron lamp-posts on concrete pedestals are placed around the structure. One of the main traveled roads of the Yuma Indian Reservation passes around the end of the canal at this point.

The construction of the intake was carried on with the same plant as that used in building the shaft. The foundations were excavated by Fresno scrapers and by hand. The foundation and floor were reinforced with old rails. The proportions of the concrete for the foundations were 1 : 3 : 6, and for the remainder of the structure, 1 : 2 : 4, granite screenings and sand being used for the finer material. The sand and stone were shoveled directly from the stock platform into the measuring hopper of the mixer; and the concrete was dumped into a 1-cu. yd. tip-bucket and conveyed to the forms by the derrick. Steam for the mixer and derrick was furnished by a 30-h.p., vertical, tubular, wood-fired boiler. The face forms for the warped walls were built on the ground in one section, 11 by 20 ft., of 1 by 6-in. tongued and grooved lumber, with studs about 3 ft. apart. They were lifted and carried to place by the derrick, and sprung to line and braced. Cut-off walls were carried 4 ft. into the ground, at the ends of the walls and at the edge of the floor.

The intake was completed in February. No additional work was necessary at the California shaft, except to remove the water and sand filling, and the temporary wooden bulkhead, described previously, and to make the connection with the tunnel.

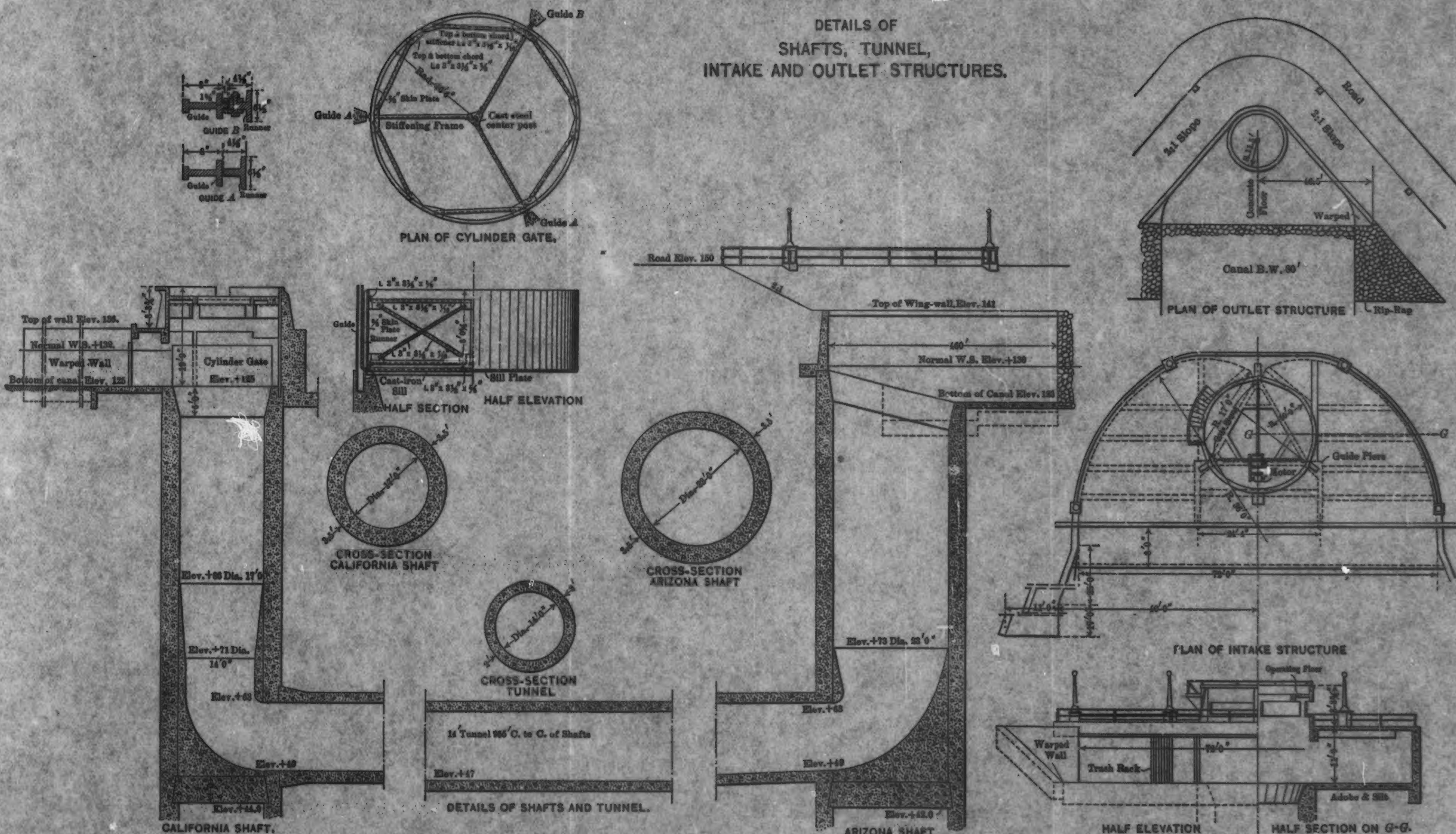
Fig. 11 shows the placing of the cylinder gate on the shaft, and Fig. 12 shows the completed structure before water was turned into the main canal.

Outlet Structure.

The outlet structure is of much simpler design, there being no control of the flow at this point. The construction, however, was more difficult, owing to the topography, the want of space, interference with the tunnel construction and the plant, and the rise in ground-

PLATE II.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVIII, No. 1281.
SCHOBINGER ON
COLORADO RIVER SIPHON.

DETAILS OF SHAFTS, TUNNEL, INTAKE AND OUTLET STRUCTURES.



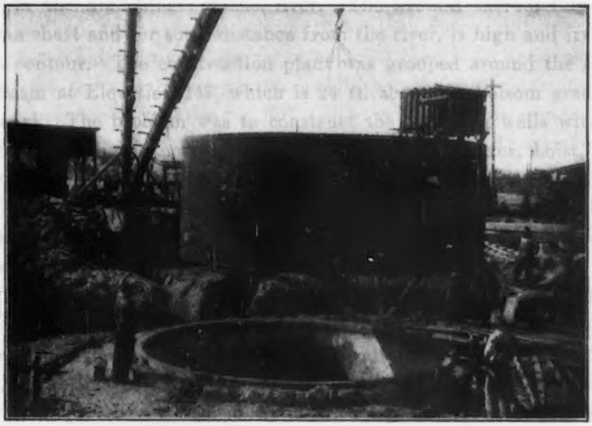


Fig. 11.—Intake Structure. Placing Cylinder Gate.

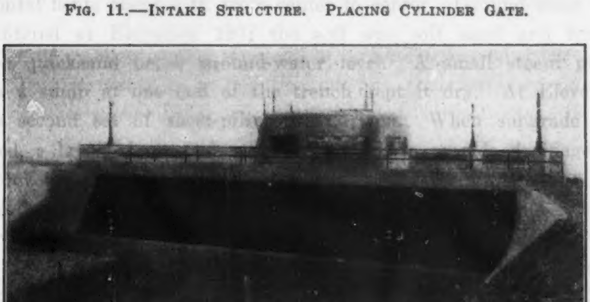


Fig. 12.—Intake Structure Completed.

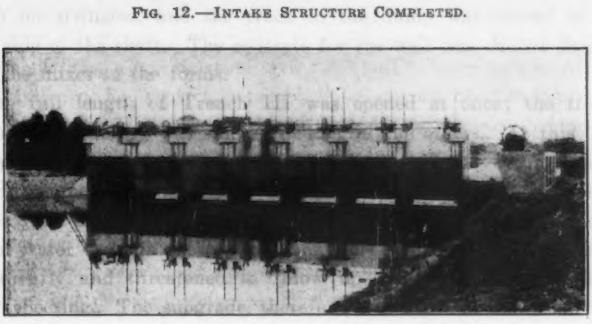


Fig. 13.—Main Canal. Check and Wasteway.

water at the high stage of the river. The ground surrounding the Arizona shaft and for some distance from the river, is high and irregular in contour. The construction plant was grouped around the shaft in a basin at Elevation 147, which is 24 ft. above the bottom grade of the canal. The problem was to construct the retaining walls without interfering with the operation of the compressors, mixer, hoist, and boiler plant. The diagram, Fig. 14, showing the layout around the shaft and the outlet, will make clear the method of construction, which was to build the walls and foundation in a trench, the core being removed later.

The east wall was built in two sections, numbered I and II on Fig. 14. Each section was about 30 ft. long and 15 ft. wide. The bottom of the foundation grade was at Elevation 116.5 in I and at 118.5 in II. Sheeting, of 2 by 12-in. planks 20 ft. long, was driven as the excavation proceeded, with 6 by 8-in. rangers and cross-braces in horizontal bents about 5 ft. from center to center. Ground-water was encountered at Elevation 120; the soil was soft sand and tended toward quicksand below ground-water level. A small steam pump set in a sump at one end of the trench kept it dry. At Elevation 127 a second set of sheet-piling was driven. When subgrade was reached, a layer of crushed rock was placed, to provide drainageway and prevent the bleeding out of the sand under the wall by the flow of the ground-water. The reinforcing steel for the wall and foundation was then thoroughly wired in place, and the foundation concrete placed. Forms for the wall were built in 5-ft. lifts, the trench bracing being removed and back-fill being placed as the wall was carried up. The track from the mixer to the shaft was carried over the trench on stringers, and the track to the dump was moved to the other side of the shaft. The concrete for the wall was chuted directly from the mixer to the forms.

The full length of Trench III was opened at once; the trench bracing and sheeting was similar to that in the others. At this time the spring floods of the river had started, and the accompanying rise in ground-water was more than 5 ft. It was found impossible to carry the subgrade as low as on the opposite wall, as the upward flow of water through the ground grew too large in volume to handle conveniently and threatened to "blow up" the soil under the ends of the sheeting. The subgrade, therefore, was made at Elevation 119,

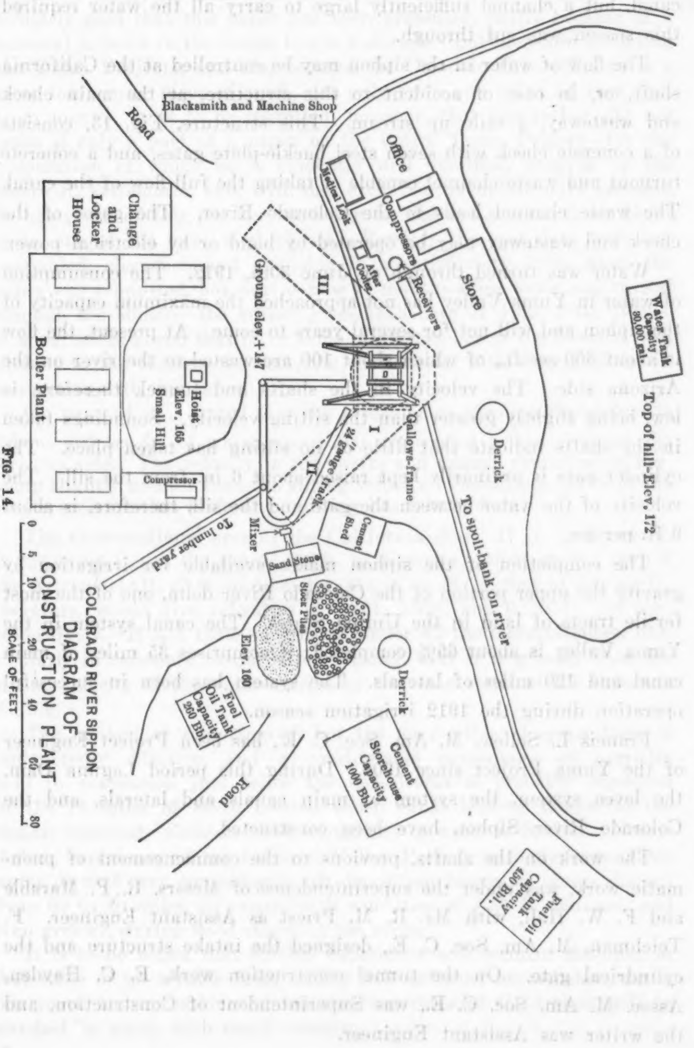
and the foundation and walls were constructed as on the opposite side. As this subgrade is 4 ft. below the flow line of the canal, there is small likelihood that the water will scour to this depth. The flow of water through the siphon during this irrigating season has been only a small portion of the full capacity of the structure, and the velocity at the outlet has been relatively low. During the present winter season a concrete floor will be placed on the canal bottom at the outlet, protecting the foundations against scour at full flow.

The retaining walls are designed to resist an uplift due to a 6-ft. head of water, combined with a pressure on the back of wall due to an angle of repose of 17° 30′. The wall is 3 ft. thick at the base, and has a batter on the back of 1 in 12. The top is at Elevation 141. From the top of the wall the ground slopes back at a 2: 1 slope to Elevation 150. There is a berm 16 ft. wide for a road, and back of it a gravel and sand hill.

The work remaining to be done from the Arizona shaft was the removal of the tunnel floor, the lock, and the bulkhead, the cleaning up of the tunnel, the removal of the cage-guides, gallows frame, and cribbing from the shaft, and the construction of a quarter-turn gooseneck at the bottom of the shaft, enlarging the diameter of the tunnel to that of the shaft.

The shape and dimensions of the gooseneck are shown on Plate II. The changes in direction and velocity are easy, and there will probably be no great loss of head at full capacity. Circular forms were built on radial lines, and the lagging was brought up and the concrete placed in 3-ft. or 4-ft. lifts. The last work done before the running of water was the removal of these forms.

On the ground between the retaining walls, where the canal leaves the shaft, there were the hoist, boiler plant, one small compressor, and the change room. The hoist was removed, and a small hoist was set temporarily on the opposite side of the shaft. The boiler plant was needed only for the derrick and hoist, as at this time the pressure was being lowered in the tunnel, and the compressors had stopped. The office was removed, and a temporary installation of three boilers was made at this point, and everything was removed from the canal right of way. The excavation of the canal had been proceeding from the other end of the heavy cut section with Fresnos and wheelers, and the short piece remaining between the completed portion and the shaft



COLORADO RIVER SIPHON
DIAGRAM OF
CONSTRUCTION PLANT

FIG. 14.

was now removed. Owing to the rise in ground-water it was impossible to excavate lower than 2 ft. above the final bottom grade of the canal, but a channel sufficiently large to carry all the water required this season was cut through.

The flow of water in the siphon may be controlled at the California shaft, or, in case of accident to this structure, at the main check and wasteway, ½ mile up stream. This structure, Fig. 13, consists of a concrete check with seven steel buckle-plate gates, and a concrete turnout and waste channel capable of taking the full flow of the canal. The waste channel leads to the Colorado River. The gates of the check and wasteway may be operated by hand or by electrical power.

Water was turned through on June 30th, 1912. The consumption of water in Yuma Valley has not approached the maximum capacity of the siphon and will not for several years to come. At present, the flow is about 300 sec-ft., of which about 100 are wasted to the river on the Arizona side. The velocity in the shafts and tunnel, therefore, is low, being slightly greater than the silting velocity. Soundings taken in the shafts indicate that little or no silting has taken place. The cylinder gate is ordinarily kept raised about 6 in. from the sill. The velocity of the water between the gate and the sill, therefore, is about 9 ft. per sec.

The completion of the siphon makes available for irrigation by gravity the upper portion of the Colorado River delta, one of the most fertile tracts of land in the United States. The canal system in the Yuma Valley is about 65% completed; it comprises 35 miles of main canal and 120 miles of laterals. The system has been in successful operation during the 1912 irrigation season.

Francis L. Sellew, M. Am. Soc. C. E., has been Project Engineer of the Yuma Project since 1906. During this period Laguna Dam, the levee system, the system of main canals and laterals, and the Colorado River Siphon have been constructed.

The work on the shafts, previous to the commencement of pneumatic work, was under the superintendence of Messrs. R. P. Marable and F. W. Hall, with Mr. R. M. Priest as Assistant Engineer. F. Teichman, M. Am. Soc. C. E., designed the intake structure and the cylindrical gate. On the tunnel construction work, E. C. Hayden, Assoc. M. Am. Soc. C. E., was Superintendent of Construction, and the writer was Assistant Engineer.

DISCUSSION

H. T. Cory, M. Am. Soc. C. E. (by letter).—The writer is particularly glad that this paper has been presented, partly because of a personal interest in the entire Lower Colorado (and therefore including the Yuma Project), and also because the Colorado River Siphon is a most interesting piece of work, not only from one point of view, but from many. It is to be regretted, however, that the author did not make it only a part of a paper on the entire Yuma Project—for the preparation of which he is evidently well qualified, judging by the clear-cut, systematic, and, with one exception, comprehensive account, he has given of this portion—and that he did not say anything about costs, one of the most vital phases. Possibly the cost data were not assembled, classified, and available in a satisfactory form when the paper was submitted, but as it is now 11 months since water was turned through the siphon—the author gives such date as June 30th, 1912—it is earnestly hoped that he or some other member of the Service, will present to the Society the complete cost sheets, with distributions, in connection with this paper. Mr. Cory.

On page 34 it is stated that the present flow through the siphon is "about 300 sec-ft., of which about 100 are wasted to the river on the Arizona side. The velocity in the shafts and tunnel, therefore, is low, being slightly greater than the silting velocity. Soundings taken in the shafts indicate that little or no silting has taken place."

The cross-sectional area of the California shaft, 17 ft. in diameter, is 226.98 sq. ft.; of the tunnel, 14 ft. in diameter, 153.94 sq. ft.; and of the Arizona shaft, 23 ft. in diameter, 415.48 sq. ft. A flow of 300 sec-ft., therefore, would give velocities of 1.321, 1.936, and 0.722 ft. per sec., respectively. It is a little surprising, under all the circumstances, particularly in reference to the latter velocity, that it is greater than the silting velocity of the water carried by the main canal at the Colorado Siding.

The method of removing all but the finer silt at Laguna Dam is undoubtedly effective, and a considerable percentage of particles held in suspension just below the head-works is probably deposited in the 14 miles of 80-ft. bottom width canal leading thence to the siphon, especially while so small a quantity of water is flowing. Nevertheless, experience in the Imperial Valley Project, with the same river water, is that very considerable silt deposition occurs at points distant from 60 to 80 miles, in canals with velocities of 1.35 ft. per sec., and even greater, during flood stages in the river.

It is to be hoped that further experience will confirm the opinion that little or no trouble from silting will develop; nevertheless, for a few years, the operation and maintenance costs of the siphon will be watched by many with much interest.

Mr. Cory. The writer also hopes that the discussion will bring out the considerations and data which caused the adoption of the various diameters of the Arizona shaft, the tunnel, and the California shaft, and particularly as to the weight assigned, in so doing, to the silt problem.

Mr. Schobinger. GEORGE SCHOBINGER, ASSOC. M. AM. SOC. C. E. (by letter).—The danger of the siphon silting up is hardly as imminent as Mr. Cory fears. Although it was operated throughout the irrigation season of 1912 with a flow rarely exceeding more than 300 sec-ft., no diminution in efficiency was noted. During the season of 1913 the flow has sometimes been as high as 600 sec-ft. with similarly satisfactory results.

Fig. 15 shows the rating curve for present operating conditions. As noted thereon, the head available may vary under ordinary circum-

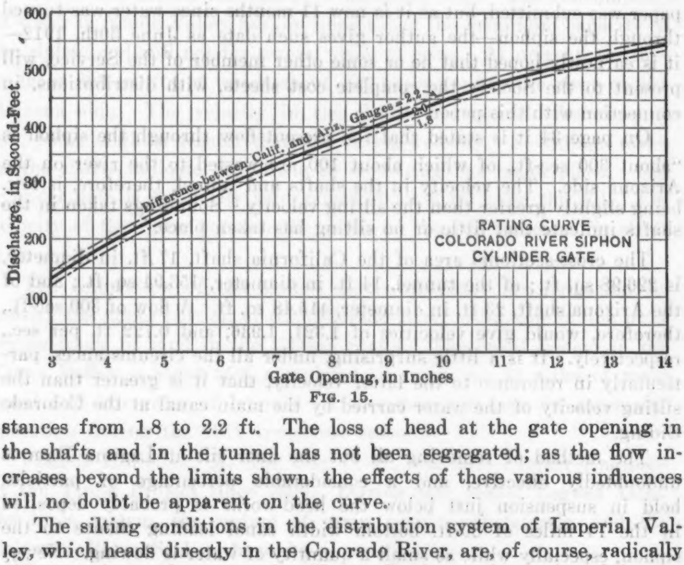


FIG. 15.

stances from 1.8 to 2.2 ft. The loss of head at the gate opening in the shafts and in the tunnel has not been segregated; as the flow increases beyond the limits shown, the effects of these various influences will no doubt be apparent on the curve.

The silting conditions in the distribution system of Imperial Valley, which heads directly in the Colorado River, are, of course, radically different from those on the Yuma Project, where the sluiceway settling basin of Laguna Dam serves efficiently to eliminate much of the silt and heavier sands, a result which was anticipated in the design of the dam. The results of a series of tests conducted by the writer indicate that from 30 to 60% of the silt carried by the river is dropped before the water enters the canal, and that more than 95% of the silt in the canal water below the head-gates is still in the water after it has flowed through 15 miles of canal at low velocity and

through the Colorado River Siphon. The velocity in the Arizona shaft is three times as great as that in the sluiceway at Laguna Dam, and therefore it is perfectly evident that a large proportion of the silt which might be deposited at these low velocities has been taken out. Mr. Schobinger.

However, even if these conditions were not so, and the unexpurgated waters of the Colorado flowed through the siphon, the writer is of the opinion that the method of operation would obviate any serious inconvenience. At the end of each week the water is shut off at the head-gates, and the canals are allowed to drain. When the water is again turned in, the head available to push it through the siphon may be as much as 9 ft., and this would effectually flush out any light deposit of the previous week.

The writer has recently taken soundings which indicate that there need be no apprehension as to the loss of usefulness of this structure due to silting.

It is doubtful whether a general article on the Yuma Project will be written in the immediate future. Laguna Dam, both in design and construction, has been described very completely by E. D. Vincent, M. Am. Soc. C. E., Resident Engineer,* and the levee work and bank protection by Mr. Sellew.† The Colorado River Siphon has been covered in the present paper, and in an article by Mr. Sellew.‡ Until the remaining important features on the Project are under way, it is not probable that a general article on the subject will be forthcoming.

The writer regrets that there has been no discussion on this paper by engineers familiar with pneumatic tunnel work.

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* *Engineering News*, Feb. 27th, 1908, Feb. 9th, 1905, and June 10th, 1910.

† *Engineering News*, Feb. 15th, 1912; *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 1482.

‡ *Engineering News*, August 29th, 1912.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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TRANSACTIONS

Paper No. 1282

THE PHILOSOPHY OF ENGINEERING.*

By Maurice G. Parsons, Jun. Am. Soc. C. E.

With Discussion by Messrs. Lewis M. Haupt, Charles Kirby Fox, A. H. Markwart, Morgan Cilley, and Maurice G. Parsons.

Introduction.

Notwithstanding that engineering structures have but recently come to be designed with precision, and that machinery as we know it to-day is a development of the past century, some of the greatest engineering feats were performed almost before history began. The Chinese wall, the Indian temples, the sphinx—the seven wonders of the world—will always compel our attention and baffle our imagination. In truth, the ancients were builders of no mean structures, but, except to some extent in irrigation, their real engineering ends there. They were not producers of economic necessities as we are to-day, but confined themselves to the erection of monuments, as such.

Later, during the Roman era, the need of political security, public utilities, and the spirit of commercialism came to be of more moment than monarchical whim, and resulted in the construction of highways, aqueducts, and sewers, which, again, command our admiration and respect, the more deservedly because they were for so long unsurpassed. Furthermore, it is remarkable that the next achievement, the reclamation work in Holland, is not overshadowed by an undertaking conforming more closely to these precedents, the fundamental cause for such a departure doubtless being economic necessity.

* Presented at the meeting of June 4th, 1913.

All this time the engineer was, primarily, a builder without special training—with nothing but his judgment and common sense as guides. It remained for the Renaissance, with the steam engine, to produce the mechanic and the technical man busied with the construction of works of public utility, and for the present day to evolve what may be called the new engineer: the man of commerce, industry, business, who makes engineering a means rather than an end—the man of affairs.

Henceforth, managers and business men will more and more be chosen from among those engineers who can go back of stresses to the money side of things; and bankers are already relying on engineers to decide the advisability of investments, the kind and magnitude of structures, the operating and financial programme. For the engineer who can go still farther back—back of the dollar, to the public good—there opens up the vast field of government administration and policy forming—the field of the engineering economist—for government, particularly of municipalities, is in large measure of an engineering business nature. This is illustrated by appraisement work, especially, as is frequently the case, if it be in behalf of a political organization, where the appraiser must decide such questions as fair profit, what to do about franchise value, whether expenditures were justifiable, what class of service is adequate, and whether to insist on extending the right of eminent domain to public utility corporations.

Considering the opportunities newly presenting themselves to the engineer, there seem to be certain professional duties which we must discharge before we may walk worthy of the calling wherewith we are called, two of these being the obligations of contributing to education (of both the public and the Profession) and of applying newly established principles.

There should be no foundation, no possibility, for a remark that the engineer should be limited by the bounds of stresses and dimensions, and not stroll over into financiering and managing. Rather, it should be a matter of common report that the engineer, seeking the truth (as he must) stands not on the shows of things but on things themselves, that he, being closely related to all industrial adventures, must of necessity be familiar with questions governing their inception and their *modus operandi*. Over and above more generally establishing the engineer's fitness to sit in councils—literally based on this capability—is a higher duty, which will become more and more clearly

defined. This is the duty of reconciling business and the people, of being a peacemaker between those who produce our economic goods and those who consume them at what seems too high a cost. Many of the differences between capital and labor arise solely from misunderstandings, ill-selected points of view, and limited knowledge: to know all is to forgive all. There are a fortunate few who, in managing and evaluating, have opportunity to establish amicable relations between seller and buyer. There is a much greater number who can accomplish somewhat the same result by participating in professional discussions, local public meetings, and consultations of boards of directors, to the extent of explaining the complexities of modern sociology and commercialism.

To the end that members of the Profession may the more ably carry on this great work of public education, they should endeavor to educate one another by systematic and thorough interchange of ideas on such subjects as business and banking, human nature and government, rather than continue to leave each to strive as best each may, obtaining by individual effort a fragmentary knowledge thereof. To illustrate this point, it may not be out of place to call attention to the confusion existing in regard to "Going Value," "Water," and "Development Expenses," as brought out in the discussion of the paper by Henry Earle Riggs, M. Am. Soc. C. E., entitled "The Valuation of Public Service Property,"* and to the fact that no one seems to know just what to do with "discount on bonds," a question discussed in parts of that paper. Such a principle that good enough is best, that "one may get too much for one's money"† should be so established among engineers as to be applied habitually. Another example of the evil results of having centered perhaps too large a proportion of attention, as a Profession, on technical details, and too small a proportion on the working principles of big things, is the unfortunate statement that "it would be, not only bad engineering, but bad business,† as if engineering and business were not, fundamentally, one and the same thing—bad engineering can be nothing else than bad business.

The second large professional duty, that of taking the initiative of applying principles, is readily suggested by two statements‡ on

* *Transactions*, Am. Soc. C. E., Vol. LXXII, pp. 255, 276, and 187.

† "The Water-Works and Sewerage of Monterrey, N. L., Mexico," *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 581.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXII, pp. 246 and 282.

engineering and economics in the discussion on Mr. Riggs' paper, one to the effect that engineering occupies a part of the subject of economics, and the other that "the engineer is essentially an economist." Without quibbling over the question whether economics is a part of engineering or engineering a part of economics, it may be stated axiomatically that modern engineering is based, not on personal ambition, national pride, worship of stone, or military necessity, but on economics. A real political economist acts on this theory. Financiers and statesmen always have been such, and engineers now are. More than this, Courts at times require them to establish their theory. Our present social and commercial structure places on the engineer the obligation of taking the lead. The engineering economist, the new engineer, must guide the banker as well as build and operate his physical plants; he must act as mediator and policy former in questions of state; he must launch out into the deep. To quote from the Presidential Address of John A. Ockerson,* Past-President, Am. Soc. C. E.:

"There seems also to be a disposition to avoid participation in the discussion of public questions, even when closely related to the work of the Profession. When Congressional Committees call on the Society for advice with regard to pending legislation, involving questions relating to engineering, it would seem to be a proper function of the Society to render such aid as may be practicable.

"In fact, it might be well, under proper conditions, to go even farther, and use the influence the Society may have to mould public opinion along lines free from local or political bias, when our public works are the subject of discussion.

* * * * * * *

"As a matter of fact, the Society is already regarded by the public so highly that its members are looked on with special favor by the Courts when expert testimony is required, by the Government when seeking for capable men for service on public works, and by municipalities where men of integrity and ability are looked for to fill positions of trust relating to the engineering side of city government.

"In a speech at the Society House recently a leading politician of New York announced that they have come to realize that the interests of the city are best served by appointing engineers to fill the important offices which have charge of the physical welfare of the city in general.

"This is true of many of our cities, and the Profession is steadily growing in public favor; loyalty to the Society on the part of all its

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* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 1034.

members wherever they may be located will greatly stimulate this growth."

Led by his interest in this newly conceived field of professional activity, the writer begs to present some reflections on sociology, commercialism, and the theory of engineering expenditures.

Social Structure.

As a foundation for the ideal conduct of life, there has been formulated the proposition that the greatest good possible should be striven for (whether this eliminates individual, genus, or species); that the common weal is the proper goal; and that, not only one generation or age is to be considered, but that all future generations and ages should have a proper weight in the final sum total of welfare. That is to say, all life has its standing in court, be it plant or animal, present or future.

To the end that man should have his fair share of this world's goods, there were organized the primitive protective leagues which have developed into our modern governments, one of the highest functions of which is modern communistic protection, the exercise of police power, defined as:

"That power which inheres in legislation to make and enforce all manner of reasonable regulations and laws to preserve the peace, order, and safety of society, and to prescribe the mode and manner in which every one may so use and enjoy that which is his own as not to preclude a corresponding use and enjoyment of their own by others."

To-day, man no longer needs protection against his original savage enemies. They are conquered, and he now turns his attention to other pursuits, foremost and basic of which is the production and distribution of more economic necessities and luxuries, the direction of "the great sources of power in Nature for the use and convenience of man." As mutual protection undoubtedly influenced the form of early government, we have, to-day, inevitably, a business form of government. Business is the chief occupation of the average individual. We live in a commercial age. Business and government to-day are inseparable, but, as originally, human welfare (and not government or business) is the *causa causans* of government and business.

This point deserves emphasis. The good of the people is the primary object of government and business, "business" prosperity being a sec-

ondary and incidental end, or, more correctly, simply a means to a richer and fuller life. The long-time security of investments, ultimate stability of government, and eventual greatest common weal of man are inextricably interdependent. Whatever truly furthers the one furthers all, and whatever threatens one does, in the long run, threaten all.

As the creature and creator, the servant and master, of this triple alliance (that of people, government, and business) stands the engineer. An abhorrer of greed, a seeker of the truth, a designer, builder, and operator of public utilities, he is at once: the people's protector, for the innocent investor can know at first hand nothing as to the security of his savings; the business leader, for a going concern is an engineering product; the advisor to governments, for business rightly has a voice in government, and many governments now embark in business; and the mediator in disputes arising between people, business, and government.

The actual state of affairs is a long way from that just outlined as the one to be striven for, albeit the general trend is toward, rather than away from, perfection: though not as yet, but some day, the lion will lie down with the lamb.

People collectively and individually contribute to imperfections; lacking knowledge of what constitutes the supreme good, man has builded on the philosophy of the good of man. Individually, we in all cases (to a greater or lesser degree) place the good of self first; one's self is paramount to one's fellows; altruism, the spirit of service, comes second. Collectively, we at times make unwarranted expenditures of money; we dissipate the property of posterity with a spirit of absolute indifference to the future; we war among ourselves, one institution against another.

Government, lacking the one-time incentive to the protection of the governed from external enemies, has given a part of its energy to protection from internal enemies. We now have vested interests and special privileges. Our economic ills of to-day, such as high tariff, monopoly caprice, concentration of wealth—and so on down a painful and familiar list—are laid at the door of government, apparently without realization that in America the People is the Government. To have nationally, we need individually, a more awakened spirit, so that our business form of government will the sooner be

purged of its ills through the application of a governmental and commercial unified policy favorable to all interests.

Business, blinded by the dust of extensiveness, has at times failed to perceive its primary fundamental object, and has been guided by its secondary fundamental object: the welfare of the people has been incidental to the apparent welfare of business. With the cessation of territorial annexation and colonization is coming the more clear-sighted and deliberate period of intensiveness, so that already the short-time balance of accounts gives way to the long-time reckoning: corporations are, happily, changing their attitude—they are beginning to see farther than their noses, and consequently are considering the effect their policies have on the ultimate stability of business through their effects on virility.

Engineering Expenditures.

As engineering enters so intimately into every phase of human life, and as every engineering proposition must needs be financed, the writer is much interested in the general subject of engineering expenditures. He realizes that:

1.—In the work-a-day world, the philosophy of the good of all life is entirely too abstract and general—it is a sort of sacred standard. There is needed a more concrete and particular criterion—a field tape, as it were—in determining whether an object be worthy, which test must necessarily be, "Will it pay? How much money will it make?" The sum total of all our investments, in order that we may not deplete what capital has been handed down to us, must net at least zero; and, if we are to bequeath as much property *per capita* as we received, this sum total must net a positive quantity.

2.—By happy combinations of capital and labor we are enabled to add to the world's wealth, its increase being measured by the profitableness of the venture which depends on the acumen displayed in selecting a field, erecting a plant, and managing the business as a whole. Each of these three phases (selection, construction, operation) is becoming, through our division of labor, with its consequent concentration and specialization of effort, multiplication of possibilities, and increase in size of units, more and more difficult of proper solution.

3.—The successful engineer is the one who grasps the principle that engineering is business—that good engineering is good business—and solves the questions, "How much?" and "Will it pay?" with capable engineering business judgment. In any engineering business possibility, the engineer, before passing on the security of an investment, must know the balance between gross annual expenditures and gross annual receipts; the difference giving the net annual income, positive or negative as the case may be, but always positive except when people are in business for their health or amusement. This rigid requirement, that an undertaking must be commercially feasible, strangles the industrial application of many scientific possibilities.

Notwithstanding that an analysis of profit, to be complete, must comprise an enumeration of the factors of gross income on the one hand and of gross expenditures on the other; and, further, notwithstanding that receipts and expenses are generally interdependent, this paper deals but little with the former. It is hoped that some one will present a paper on the elements entering into gross income.

In brief, total income is the product of unit prices and the number of units sold, both fluctuating according to the law of supply and demand, or the law of monopoly distribution—in any event according to the law of variation of returns.

The factors of total annual cost—to deal with this side at somewhat greater length—are interest on first cost (unmistakably including development cost); depreciation or sinking fund; the creation of a reserve for extensions, betterments, and unforeseen contingencies; outlays for raw material and supplies; maintenance and repairs; wages, salaries, and royalties; rent, taxes, and insurance. These factors, intricate within themselves, are, in addition, involved in a complex interdependency. To illustrate:

In designing an important bridge, considerable attention may be given to the question of economic dimensions, first cost alone being considered. It is then to be remembered that first cost and maintenance are bound together, indifferent material and workmanship necessitating heavy up-keep charges. Moreover, precisely how fast depreciation should be allowed to take place and exactly to what extent it should be checked by repairs, is only one of the multifarious issues the engineer must determine. To wander back, for a moment, to receipts, the rate of exploitation of an exhaustible resource is of fundamental

importance in its bearing on first cost: Should a plant be installed capable of using all available raw material in ten years, or a thousand? As a final example, illustrative of the influence any factor of the cost of production has on every other factor, and of the weight each has in the balance between income and outgo, we may consider the rate of exploitation of a perpetual resource; more specifically, we may ponder on the height of dam in any water-supply problem as it enters into questions of cost and useful and wasted draft.

These considerations bring us to the general question of the best size of engineering expenditures ("What is the best height of dam within the limits of height giving rise to a proper return on cost," and "What is the best rate of exploitation of a limited quantity of ore," being two examples), on which question the following theory is formulated:

Of all possible arrangements of all expenditures, that one is best which secures the maximum present worth of ultimate total incomes. This statement applies to an imaginative case dealing with the sum total of all expenditures and receipts. That is to say, it does not mean that if $10 000 000 spent on a railroad will net a final profit worth now $15 000 000, whereas $20 000 000 spent will yield a sum worth now $26 000 000, then the latter amount is the better to invest, unless the second $10 000 000 would be idle or be put in an enterprise which would ultimately return a sum the present worth of which is less than $11 000 000. To generalize this limitation: all possible opportunities for investment should be considered in the light of all the revenues they would yield, and those investments should be made which would secure the maximum total of the present worths of all incomes, more or less capital being put here or there to procure this result. The example of the dam may further clarify matters: From the point of view of the dam investment alone, we want the highest rate of interest; but it may be possible, on the one hand, to put elsewhere part of the money available for this purpose at a yet higher rate, in which event we would choose some other than the highest rate obtainable from the dam; on the other hand, a dam larger than that securing the highest rate on the dam cost might yield more returns on the surplus investment than that same money could earn elsewhere, whereupon we would choose, again, some other than the highest rate obtainable from the dam. In other words, we would put our money where it would do the most good.

Time is another element included in the general statement, as may be seen from the following example: If to exhaust a body of ore this year would net 0.0872x$ while to save that ore for use during the fiftieth year from now would net more than $$x$ (interest being taken at 5%), it would be good business thus to delay, $0.0872 being the present worth of $1 fifty years hence. Or, again, policy would dictate delaying 50 years if then more than 11.4674y$ and now only $$y$ would be produced (interest being at 5%, $11.4674 is the value of $1, at annual compound interest, fifty years hence). Taking this same body of ore, we could mine it all in, say, one or ten years. If (disregarding other places to put money), beginning now, we mined it in one year at a net profit of $$z$, and by mining it during 10 years at a uniform profit we obtained an annual net profit greater than (1.62889 × 0.0795) $$z$, or approximately 0.1293z, it would be the part of wisdom to exploit at the slower rate. (Here interest has been taken at 5% per annum, $1.62889 is the amount of $1, at annual compound interest, ten years from now, and $0.0795 is the annual annuity required to accumulate $1 in ten years.) In other words, we would spend our money when, and at the rate at which, it would do the most good.

To repeat: The ideal engineering expenditures would secure the maximum present worths of net income.

To obtain this ideal distribution of financial resources among all departments of industry would necessitate two steps. Of these the first would be a set of differential investigations (for each and every commercial enterprise), enquiring into the relation between expenses and revenue, several trial solutions being necessary to determine the results of a variation in the size of physical plant and of a postponement in initial development or subsequent improvement. These differential investigations having been made from time to time, and as new possibilities presented themselves, the results would be sent to a central clearing house where the second step, that of integration, would be taken. There the higher engineering would be performed—the financial decisions made—in the light of public policy, industrial and human welfare, and the results disclosed by the differential investigations. From a purely commercial point of view, effort would be put forth to secure such an extensity and intensity of industrial activities, by introducing changes here, allowing more money for construction or operation there, hastening exploitation in one place, and

conserving resources in another, as to secure constantly, under changing conditions, the maximum present worth of net incomes ultimately derivable from all business. The men who would thus fix the world budget should necessarily be broad and sympathetic, of absolute integrity, well schooled and experienced, and possessed of such a knowledge of the inwardness of things as to realize that he who would rule must be servant of all, and to base their apportionments on the proposition that, in the end, the labor and capital interests, the human and industrial interests, the interests of people, government, and business, are one.

In the commercial world we could stub along without a realizing sense of this underlying theory of engineering and business, of engineering expenditures. Indeed, we seem in cases to be absolutely uncognizant of the relation between business and people, or of the principle of compound interest. Some concerns muzzle the ox that treads out the corn, others fail to conserve resources, and others, contrariwise, are admittedly charitable. For the most part, however, our organizations fit into the system in a fairly satisfactory way, from the point of view both of the humanist and the commercialist. The secondary fundamental of business, that a concern must make money, is automatic in its application. If a business does not pay, it fails—the result is simple and unavoidable. We must get more money out of business, as a whole, than we put in (financial efficiency, so to speak, must be apparently greater than 1), and the only way to do this is to turn labor into capital by means of existing capital. Our machinery will never be completely self-acting. We, also, must work. In striking a balance between gross annual cost and gross annual income, there is, in all organizations, considered as a whole, a positive profit, except in early or bad years.

In the case of large and old companies, this profit is at times less than it should be by reason of unknown development costs. Nor is development cost the only factor of total cost sometimes neglected, depreciation being overlooked only less frequently than it; but, all in all, any going concern pays. If too many items are neglected, one way or another, it ceases to be a going concern.

Only by chance can the best arrangement of all factors of total cost be had: Proper maintenance is dependent on first cost, length of life, cost of repairs, and cost of reproduction; operating expenses

are subject to change without notice, because of improvements in the arts or fluctuations in supply prices or wages; accident frequently plays its part in the cost of the finished product. These and many other influences render the minimizing of costs a comparative matter requiring the best of engineering business judgment.

In the daily conflict of the commercial world, there is a strong temptation to overlook the theory of maximum present worths of net income (which, it may be noted, is at the bottom of conversation), to forget those generations yet to come, and to strike out, every fellow for himself and "the devil take the hindmost," in pursuit of maximum present returns.

This is due in some instances partly to selfishness, but, regardless of motive, there are always certain circumstances limiting the perfect adjustment of engineering expenditures. One of these is our lack of information of the future: we know not what a day may bring forth. Another is the fact that some people prefer to have their property within sight than to place it elsewhere more profitably. Again, many do not wish "to carry their eggs all in one basket," even though a single one promises less trouble and more gain. The fact that capital must occasionally completely develop or leave entirely untouched property considered as a legal entity, whether or not the maximum possible development is the most economical, whether or not a part of the required capital could be more profitably placed elsewhere, is still another circumstance limiting perfect adjustment of expenditures, as is also the requirement that certain funds be handled as a financial entity (be kept intact) even though a distribution of means between two or more projects would bring better results. Lack of funds is most common of all. The foregoing classes of limitations may be considered as special cases, and are unimportant in comparison with the general prevailing incomplete knowledge of present and future conditions and possibilities. The man with only a small amount to invest cannot make large outlays in search of the best opportunity, but must "hit a head when he sees it." Even those who handle large sums can, as a rule, be informed only in certain specialized fields of business, and cannot completely cover even those; and at best the heaviest bankers can know of only a few of all the places where they can make secure investments.

This absolute impossibility of sending all information to a central

clearing house, taken with the accompanying red tape, which would tie up large amounts in investigations, and in which human frailty would certainly become entangled, has given rise to local and national concerns; to a practical working engineering business organization composed of small and large units, apparently separate and distinct, but, in effect, part of a unified whole.

Small rule-of-thumb undertakings may require no engineer whatever, but the small banks and, as a rule, individuals controlling more than very moderate sums, retain engineers for advice or to design as per instructions the physical plants necessary in the every-day local affairs—such affairs as are assumed to be of a paying nature and require only limited capital.

As we get into fields of more and more importance, the engineer occupies an increasingly conspicuous position, an example of the transition from local to national engineering being given by the case where an energetic citizen makes a water-power filing, then has a preliminary investigation of possibilities made, the result of which may justify careful and detailed study by more experienced men, which study may in turn lead large financial interests to design, construct, and operate a hydro-electric plant. So we lead up: an idea, a local engineering study, a bond issue development. The local engineer handles the small work; big jobs must be done by those who can furnish the money.

There is thus seen the absolute necessity for a money trust—for a ring of men commanding tremendous amounts of capital (it may be the accumulated individual savings of a nation) furnished with accurate information as to the probable balances between expenses and incomes of large ventures, endowed with good judgment, and broad in philosophy, which ring can direct where money shall be spent, how much shall be spent, and who shall spend it. A money trust, a central authority, is an absolute necessity.

Certain abuses with which "money" is now charged are not necessary: The producer should not be underpaid while the consumer is overcharged—he is the same man, and on him rests "prosperity." Arbitrary decisions should not be made contrary to the revelations of scientific discoveries, with the object of momentarily swelling the private purse. No just cause can be found for dealing as we do now in shorts and longs—for selling what we have not got and buying what

we never expect to own. Nor can we excuse violent and sudden fluctuations in the value of stocks and bonds, with the accompanying panic or strong market, as the manipulator may desire, while the wheels of industry turn, unheeding, precisely the same. Furthermore, gain resulting from high prices and low wages should not be capitalized. "Water" is bad business. The management of investment securities with resulting transfers of certificates of ownership, which may be called financial engineering, is a necessary incidental of modern economic conditions and an entirely different thing from bull and bear skirmishes. Such things are in fact entirely foreign to industry, to good business, to sound enterprises. "Such things ought not to so be."

Conclusion.

Having passed through the eras of constructing temples, of erecting personal monuments, of cutting ourselves off from intercourse with other peoples, we stand to-day in the age of international commercialism, but not of commercialism at its height. Although the pendulum seems to be swinging back, although some sense again the raising of roses instead of dollars, we have only begun to see the vast field of engineering business. Commercialism, big business, is here to stay.

The application of the theory of engineering expenditures is possible only when bankers have a broad vision and a wide range of choice; they can invest money most profitably only when they know all possibilities. Economy of production and satisfaction of service are best obtained by large units, by monopolies. Competition, with its duplication of expenses, is uneconomical, and contrary to the principle of division of labor. We want one butcher, one baker, one candlestick maker. The best kind of a business is a well-managed big business. The day of little things has passed.

As each individual now has a voice in local and National government, so too each will come in time to have an actual direct share in small and in big business. Each will have some daily occupation (something to keep him busy with his immediate physical surroundings), together with securities (bonds, or shares of stock) issued by a large business concern. We will come in time to a common ownership of big business, over and above general possession of only personal effects, either by government ownership or by widespread indi-

vidual ownership of small amounts of stocks and bonds. In either event, because of the interdependency of business, government, and people, we will come—are coming—by means of enlightened self-interest, to a closer alliance between them. Enlightened self-interest is not anti-commercialism, it is pro-commercialism. The making of money is not incompatible with the making of men. Both processes will continue. Corporation publicity and the spirit of individual service are only the first feeble gasps of a new-born co-operation of business, government, and people.

DISCUSSION

Mr. Haupt.

LEWIS M. HAUPT, M. AM. SOC. C. E. (by letter).—Mr. Parsons' admirable paper is worthy of being read and known of all men, and especially by the members of the Profession whose duties make them pioneers in the development of the resources which the Creator has placed at their disposal.

The altruistic, fair-minded, open-handed spirit which permeates this "philosophy" is so fully in accord with the Golden Rule that, if it were realized and practiced by engineers in general, it would make them the dominant agency for the general uplift of humanity. The man who is in touch with Nature and correctly interprets her secrets is, of necessity, a lover of truth, and one to inspire confidence in his judgment and integrity.

It is true that a good engineer must be an economist; but economy is not always synonymous with business. "High finance" lives on credulity, fluctuations "on 'change," cupidity, patronage, and other elements, too often created by and sanctioned under the guise of law, and it is erroneously assumed that the more money put in circulation by appropriations from the public treasury, the greater the benefits, regardless of the results. It should not be forgotten, however, that the revenues are derived from the people as a whole, and are doled out to special interests employed by or under the control of the Government, whose main purpose is to serve the appointing power and hold on to the party patronage, so that the larger the appropriations the greater the power and the stronger the party. This policy of Government soon leads to despotism and profligacy, with resulting distrust and destitution, the remedy for which is to be found in an economic, judicious application of public monies to secure definite results on approved plans demonstrated by experience, and also on free and open competition, with right of appeal—in case of disagreement—to impartial arbiters, and not to the parties who frame the specifications and make themselves the final and sole authority. Such a policy has brought ruin to many honest contractors who have met with unforeseen and unexpected physical obstacles, or with ignorant inspectors.

The author has shown how "many-sided" the well-qualified engineer should be, not only in the various branches of his profession, but in the collateral ones of finance, legislation, organization, and business in general, and not merely an honest man skilled in the technique of his calling; yet how few there be who reach this standard.

If the engineer who knows what is required in the physical world with which he has to deal does not also assist and advise the Legal Profession or the legislator in the framing of laws authorizing and regulating transportation or municipal improvements, there is great

Mr. Haupt. danger of injury and injustice being done to the community; as, for instance, in the case of the disposal of the sewage of densely populated centers like New York or Chicago.

In the latter instance, the quantity of potable water required to render the sewage innocuous was carefully determined, per capita, and an open conduit was constructed, by legal authority and enactment, to discharge that required volume across the Chicago Divide into the Illinois and Mississippi Rivers—a quantity necessarily increasing with the density of population. Now, when it is found necessary to pass 10 000 cu. ft. per sec., to dilute the sewage sufficiently, the Government withholds its permission, leading to the inference that the residents along the route of the outflow must be content to use water contaminated by sewage, injurious to health, and which must increase in its percentage of albuminoids with time; or else, that the Sanitary District must find some other method of disposing of its refuse matter after expending millions in good faith and by approval of the Government. "Business and Government" are inseparable, but the latter should be liberal, not dominant; stable, not fickle; regulative rather than executive in the physical works and operations which may be safely and honestly administered by competitive corporations.

If, as stated, the "good of the people" were the primary object of Government, partisanship would soon disappear, and many of the evils accompanying politics would vanish. We cannot expect such a condition, however, under the *laissez faire* policy of keeping out of politics and letting the machine run itself, in the old ruts. It needs the engineer to prepare the way for a better standard, socially, morally, physically, and even spiritually. Then "every valley shall be exalted, and every mountain and hill shall be made low: and the crooked shall be made straight, and the rough places plain": and an highway shall be there, on which the wayfaring man may find occupation and repose.

Mr. Fox. CHARLES KIRBY FOX, ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read this paper with much interest, and thinks that any papers or discussions which will tend to broaden the field of engineers should be encouraged.

The author's questions, "Will it pay" and "How much will it cost," reach the core.

Speaking of the old-time engineer, the author says: "All this time the engineer was, primarily, a builder without special training—with nothing but his judgment and common sense as guides." From old and new works and from plans which the writer has seen and heard about, he is inclined to think that these two items, with a liberal allowance of loyalty and "stick-to-it-ness," are the principal qualifications of an engineer. He cannot have judgment unless he has acquired a good education and has had extensive experience.

In connection with the author's general statements, it might be worth while to take into consideration the testimony given in the preface of Wait's "Law of Operations Preliminary to Construction in Engineering and Architecture": Mr. Fox.

"It is not the mere competency to design, draft, lay out, and superintend work that gives reputation to an engineer. This is work done by assistants who are comparatively unknown to the profession. The men who control and direct the work are men of broad ideas and business capacity, whom companies and proprietors expect will look after their business conservatively and hold their investments secure and profitable. This, it is contended, depends largely upon their business and legal training. Without this training graduate engineers find their many technical qualifications without weight in the estimation of their employers, and they feel it keenly when men with a general education are taken from the ranks of clerks and office help and are given direction of work as superintendents and managers wholly on account of their knowledge of the business policy which directs the financial operations and because they know from association and study how to decide ordinary questions of business and law."

Following along the lines of broader business policy for engineers, H. T. Cory, M. Am. Soc. C. E., recently presented before this Society a very interesting and instructive paper on "Irrigation and River Control in the Colorado River Delta."* He not only described the engineering features, but gave a complete financial and commercial summary of this important work. This is the first time the writer has seen this important part of a construction enterprise treated in this manner. It is now well known that the overhead charges, such as promotion, financing, discount and interest on bonds, general charges, etc., amount to from 50 to 100% of the cost of the construction items. Many papers are written on the technical side of engineering, but practically none on the business and financial side.

At one time it was considered unprofessional for an engineer to have anything to do with the contractors or actual construction. In other words, his work was confined to the survey, designs, and inspection. Now, there is hardly a contracting company which is not handled wholly or in part by engineers.

Engineering has been defined in many ways as "the art of making the dollar go the farthest," and "the art of directing the great sources of power in nature for the use and convenience of man." Why not combine these and define engineering as the art of directing the great sources of power in nature for the use and convenience of man in the most economical manner possible?

A. H. Markwart, Assoc. M. Am. Soc. C. E. (by letter).—This is a very thoughtful paper. The engineer is becoming more and more a necessary and important factor in the modern social and economic Mr. Markwart.

* *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 1204.

Mr. Markwart.

system, and such papers tend to bring out this point in a dignified and scholarly manner.

In this day, labor is of value and creates value, and the highest efficiency thereof is constantly sought to the end that there may be no economic loss. The use of labor and the expenditure of capital in an efficient manner is the function of the modern engineer. In remotely ancient times human life had but little value, and a Pyramid was conceived to satisfy a ruler's whim or furnish for him a monument by which to be known after he ceased to exist. Though it is true that the ancients constructed such monuments with considerable skill—and one wonders how and by what means they were carried out—there was no wealth created thereby, as we understand the term "wealth"—the result of human activity.

Modern engineering is the result of necessity, and offers a peaceable field for achievement. The construction of such a great work as the Panama Canal, for the benefit of humanity at large, calls forth one's unbounded admiration. The engineers on this undertaking have made a personal sacrifice. They have given time and attention to a work which isolates them for a long period, and acts to their personal disadvantage in many ways. The emolument received is small, and they are removed from their usual associates and haunts, causing them to be more or less forgotten; the gap made is soon closed. Truly there is much food for thought when one realizes the obligation of society to such fore-lopers.

The engineer of ancient days knew nothing of the science of engineering as it is now understood. His work was the result of experiment. If a wall 1 ft. thick was insufficiently strong and fell, one 2 ft. thick was provided in the next venture. Beware of the engineer of the present day who constructs his work in this manner. It is the tendency of the engineer to be, not a parasite, but a creator, and a creator with constructive and not destructive theories. Furthermore, by training, his impulses are those of the conservationist. Few engineers would knowingly waste the materials and resources at their disposal. These are tendencies in the right direction, and are fundamentally economic in theory.

Normally, the engineer has but little interest in public life or political activity. This seems peculiar when we reflect that all his training has been along a line which tends to produce the careful and logical thinker. He should be a factor in any progressive régime, whether political or commercial, as he is not hedged in by precedent or prejudice. Generally speaking, the engineer has a responsive and receptive mind on ordinary problems, and he seeks to be shown the truth.

The writer at this point would call attention to the apparently increasing demand on the engineer in the architectural problems of to-day.

It would seem that the architect is gradually and voluntarily giving up his place as master builder. He does not fill the same office as did Michael Angelo, who was in turn, sculptor, painter, architect, engineer, and poet. On the contrary, the modern architect does not allow himself to qualify in this broader field of art; he assumes his most important work to be along esthetic lines alone. He is more concerned about the exterior appearance of his building than about any other feature. The façade, color scheme, and floor plan seem to hold the major part of his attention. Whether the construction of the proposed building will be a good commercial proposition is of no concern to the architect; rarely does he make a logical and convincing analysis of conditions to be met or problems to be solved. This is not a criticism, but a statement of a condition. Such matters as the financial report, the economic design, and the execution of the scheme are frequently left to the engineer. This is in support of the general idea advanced by Mr. Parsons, and is a confirmation of the opinion that the engineer is entering a broader field of activity. Mr. Markwart.

There is at present a movement on the part of engineers to develop a code of ethics. This should be encouraged and fostered. The architect has his code of ethics, and, generally speaking, it is a recognized and established standard for the profession, even in the eyes of the layman. All such codes have, as a component part, a schedule of fees, and though, on the face of things, this may appear to be mercenary, there is no doubt that there is considerable necessity for such a feature. The engineer should be adequately compensated, the compensation being measured in terms of the service rendered. A manifestly low price or insufficient compensation is an argument for a poor article or an inadequate service. A proper compensation must needs result in a better moral standard, and any code of ethics should include a schedule of prices, to bring this about.

Mr. Parsons has mentioned the ever-increasing demand for engineers in the various political, social, and commercial fields. It is becoming more and more the function of the engineer to act as adviser in actions or disputes between labor and capital; he is becoming the scientific purchasing or selling agent for large corporations; he assists in the valuation of property, for sale or rate-fixing purposes, and he frequently takes up sociological problems of considerable moment.

Engineering, from a broad viewpoint, requires a large supply of common sense, executive ability, and action. Our colleges should take this into consideration in their teachings, together with the question of moral training, to the end that to be an engineer is to be one who can be relied on to operate successfully under a high moral standard, so that his opinion and advice will be sought when other sources fail.

Mr. Markwart.

It has been said that the engineer frequently looks after his client's financial interests better than his own. There is probably considerable truth in this remark, and such a condition is inconsistent when we consider the broad responsibility of the modern engineer. This must be avoided, for there is something in the theory that personal financial prosperity is to some degree a measure of one's success. Prosperity, to some degree, promotes higher ideals, and, after all, the higher ideal is that which should be attained in all walks of life.

The old theory that a business should charge all that the traffic will bear is false, and other advanced and broader theories are taking its place. Good service is now, and will continue to be, considered as that which is to be rendered. Competition of service will in the future be the spur which actuates opposing concerns. This principle is essentially with us and in operation. From now on, a reduction in price is not necessarily going to obtain the business. Superior and improved service will have to be rendered. The "Philosophy of Engineering" very clearly brings out this phase of modern business.

It is only a question of time, in corporation activities, when many mooted points will be settled, and very probably by the engineer. For instance, what consideration should be given in the case of free rights of way as regards rate-fixing purposes? Should the public be a partner to this extent? Again, in continual change of methods, the cost of plant equipment is being constantly reduced. Early-day plants cost more than latter-day plants, resulting in cheaper service to be furnished by the latter-day plants. Does it seem fair to "junk" the early-day plant which was built in good faith? Further, the unearned increment principle will be put on a firmer basis. All such are problems that should be solved by the engineer.

The writer disagrees with the author as to whether the "Money Trust" is an absolute necessity. The question of proper control is so complicated that it is doubtful whether all conflicting points could be consoled. A powerful money trust unquestionably destroys private initiative. Many an enterprise starts as a result of individual effort. We should do nothing to prevent such initiative, for the reason that many worthy movements would thereby remain unborn. Individual initiative must continue, and to continue there must be incentive, and to have that incentive there must be the anticipation of personal profit. New or under-developed localities would remain dormant if left to the money trust alone.

The author suggests the possibility of an economic rate of investment as regards the public at large; a commercial clearing house, as it were. This is Utopian, but is an interesting thought. It would be quite possible, however, in individual cases of investment, to determine at what rate, for instance, ore bodies should be exhausted in order to produce the most economical results, in the broadest sense,

time being considered. To obtain the ideal expenditure is a nice problem of the calculus, similar to, but more complicated than, the determination of the economical diameter of a pipe line in a power plant so that the value of power lost by friction bears the proper relation to the value of the horse-power generated. Mr. Markwart.

The writer will conclude by agreeing thoroughly with the author that the commercial or big business is here to stay, but we must temper it with the esthetic, to the end that what we do is good and great, whether it be the development of an idea or a piece of construction. Our acts and our engineering should be good to behold. Even a monument here and there, as in the time of the ancients, will do us no harm.

Morgan Cilley, Assoc. M. Am. Soc. C. E. (by letter).—This paper deals with a phase of the Profession which has received very limited thought; indeed, too little; but, of course, pressure of affairs has prevented the engineer from devoting to it the time which, otherwise, he would have liked to give. Mr. Cilley.

It is not less important than the subjects which receive more attention, and the neglect which it has suffered has been responsible to a large extent for the lack of appreciation for the engineer and his absence from the councils of higher affairs.

The neglect of kindred lines of study in college is the beginning of the engineer's mistake; he too often sees too much importance in the technical side of his course, and in after life too readily accepts the position of a servant too engrossed in technical details to be bothered with, and, as it is sometimes expressed, "to bother with," managerial councils. Ernest McCullough, M. Am. Soc. C. E., in his paper "Engineering Education in Its Relation to Training for Engineering Work,"* handles this side of the subject exceedingly well in the studies he recommends for the third-year course, and it is in the perfunctory attention to the subjects mentioned that lies the lack of appreciation for the engineer's importance in the social structure as brought out in Mr. Parsons' paper. Does he display the interest in that social structure which would inevitably draw him into the fabric of big things, or, possessing that interest, allow it to languish beneath his engrossment in details?

In fact, the engineer can learn a great deal from the salesman and advertiser. Publicity will benefit the engineer. Just as in business, so in engineering, the consumer has to know where to look for his engineering service. Among engineers there is no need of the strict code of ethics that exists among medical men. Where is the public to find the capable and efficient man, if that person keeps himself practically unknown outside a very narrow circle of acquaintances?

Then, when his opportunity comes, he is at a loss how to present his facts, ideas, and convictions in the briefest yet most telling way.

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 1090.

Mr. Cilley. To accomplish this he must have in mind the philosophy of engineering, a working knowledge of psychology, and experience in public speaking, backed by convincing assurance. In fact, he must be a salesman. An engineer from one of the larger engineering offices of New York City, one who had charge of a number of improvement works, once boasted to the writer of having, that day, gone before a meeting of a town council, in connection with his work, and made a telling speech in which he persuaded it to make certain additions to the plans. An average traveling salesman would have thought no more of that than that engineer would have thought of accomplishing a survey which was a little out of the ordinary.

Continuing Mr. Parsons' paper, we read:

"This rigid requirement, that an undertaking must be commercially feasible, strangles the industrial application of many scientific possibilities."

This is only too true, and yet the development of scientific possibilities, which may have seemed at first to be lacking in commercial feasibility, has made the Edison Laboratories the greatest center of valuable invention of history, and also has put the German nation, on account of its government laboratories and government-aided private investigations, in the foremost rank of chemical knowledge and development.

Mr. Parsons argues the importance of the development of scientific possibilities which tend toward the ultimate good of humanity, and on this bases his convictions as to the necessity of a "money trust." Then, as the government is the people, and a money trust holding the accumulated savings of a nation is a necessity, it and the people's government should be one and the same.

The passing of Mr. J. P. Morgan is conceded by all to mark the beginning of a different era of financial arrangements. The economic ills mentioned by Mr. Parsons are traceable to human frailties, and, recognizing this, the people are insisting that their accumulated savings shall not be risked in the power of one man or group of men, regardless of irreproachable character or integrity, unless he or they are directly responsible to them for their acts.

The United States Government has departments which have for their work functions in other fields, similar to those mentioned by Mr. Parsons for his central clearing house, and their acts have the power of the people's government behind them. Then why should it not have a department with the functions the author suggests?

Further, as it would require engineering of the highest order, and in which the Profession would be of greatest importance, why should not this Society, in conjunction with the other great engineering organizations in the country, take the initiative in a movement toward that

end? As engineers have become much better recognized, concerted action on their part would certainly receive attention. If recommendations are based on the deepest thought, give evidence of having been thoroughly considered, be devoid of the least traces of personal or professional ambition, and carry with them the conviction that they are for the far-reaching good of humanity, the movement will succeed. Mr. Cilley.

MAURICE G. PARSONS, JUN. AM. SOC. C. E. (by letter).—The writer's paper has fallen into the hands of men more worthy than he, who have treated it from various points of view, with the matureness of thought which comes from decades of practice, and, as masterly theses on kindred subjects have recently appeared elsewhere, little could now be added advantageously by any one. On the subject of the money trust, however, which at present incurs the disfavor of several factions, the writer desires to make some closing remarks. Mr. Parsons.

Past abuses by this elusory organization must be corrected, not from without, but from within. Only the money trust itself, prior to a much more serious stage of the malady with which the nation is afflicted, can temper its injustices. Voluntarily, when it comes under the control of men big enough to be more generous, it will allow adequate compensation to all producers; for its own welfare, private initiative will be encouraged, the man with a good commodity will be given opportunity to find a market, and will receive a fair reward. Under a stronger and more enlightened money trust, inhuman greed will be peacefully diminished; capital, as capital, will consider the substantial needs of the people, its retainers; and stock exchange juggling will cease. Through the merest common sense, it will correct its many abuses.

Moreover, the control, by a small ring, of much ready money is necessary in order that tremendous undertakings may be accomplished, that new territory can be vitalized as the result of world-wide prolonged conscientious search for that which is most promising. The present crisis of international tension, financial stringency, political and social unrest, could be passed quickly and safely if all bankers, in a spirit of mutual interest, would get together with that end in view—if there were a firmer and better money trust.

Nor is precedent lacking for such a financial authority: The Edison Laboratories are not content with the resources of any one locality. If something is wanted, it is searched for, everywhere, at great cost. As a result of world-wide investigation with central clearing-house reports and decisions, the Edison Laboratories produce handsomely for the use and convenience of man. In political government the individual is not a large enough unit; neither is the family nor the state. We have a national head and even international formalities. Similarly, the individual is normally merged financially in the local

Mr. Parsons. community. Next in order, is a sub-control of local affairs, then an accumulating of resources in some particular bank, and so on.*

Must we not have admittedly a supreme authority in matters of finance, a coterie of international bankers, with its hand on the pulse of the world?

In a broader conception, these men would be members of more than a commercial clearing house. Money is simply a medium of exchange and standard of value. It represents stored-up life, and as such is to be regarded with awe. Those who control it control the lives of the people and the welfare of the nation. All differentialities of human interest are not reduceable by man, with his human frailties and only human knowledge, to precise amounts of money. It is necessary, therefore, that these men have, in addition to ability, training, experience, and judgment, a broad human understanding. Furthermore, they must be men who can oppose with an iron hand, if need be, improper tendencies. Study should be given by them to such matters of national interest as the high cost of living, the pernicious influence on morality of rag-time thought, speech, song, dance, and dress, the fact that in so many cases *viri* has given place to *homo*. World patriots of finance, in the interests of capital, could do naught but afford an "even break" and a "square deal" to each and all, for, to secure perpetually a maximum present worth of net incomes, there must be happiness, loyalty, industriousness, integrity—all the superb qualities—on the part of all the people. Industry should be for the use and convenience of man, for, in the last analysis, it is not bank accounts, but lives and souls, which count. The engineer of to-morrow will be, let us hope, a human engineer.

It were an easy matter to enlarge on the powers of the money trust until it absorbs every function of government. We should then have to ask ourselves where we were coming out; what these all-powerful men might do; and how they would be chosen. Under a pure democracy, we should run the danger of having for our rulers a prize collection of demagogues. With a restricted democracy, we might gradually be overpowered by an oligarchy serving its own end. An inherited monarchy would sometime fall to an imbecile.

As it is to-day, we have a system that works, and no great need exists for a radical departure, although some precautionary measures should be taken. The Reclamation Service and the Department of Commerce and Labor, the one spending many millions of dollars, guided presumably by the ideal theory of investing our money where, when, and in what amounts, it will do the most good, the other serving the people in a sociological sense—these two divisions of government might unite forces, developing into a department-controlling

* The reader is referred to an article on the failure of the First-Second National Bank, of Pittsburgh, in the *Literary Digest*, July 19th, 1913.

industry, but it is hardly likely that such is a necessary step. We have now a government to look after our many needs and troubles. We have financial rulers (the real rulers) deciding our commercial issues. We have individual strong men. On occasion, they are at variance, and unavoidably so, for it is rare for even only two minds to be in perfect accord. Mr. Parsons.

What will be the new philosophy of engineering? Will there be an industry for profit, an industry for abuse, an industry for profit and use, or will there be chaos? We are facing a big problem; one for the individual, educator, and engineer to solve. Social, political, educational, moral, and economic conditions are changing. The concentration of population presents its own difficulties. With all our necessities and luxuries we clamor for more, in the belief that some day we shall have enough to make us happy and industrious. The vast resource of opportunity of the past century has passed with it. Competition from now on must be for quality. Peace prevents the elimination of the strong, sanitation and amelioration of conditions preserve the weak, so that the strong become stronger, the weak, weaker, and competition increases under an artificial selection. This means more bitter disappointment to more losers. The struggle for existence is a relic of the past, but in this game of life, what strife may come? The question is not one of organization, but of philosophy; not of how much property is accumulated, but of the use to which it is put; not of system, but of men. We need a closer harmony between existing working parts.

The big task before the engineer to-day is to help the world out of the uncertainties of complexities, theories, and tendencies—the work of his hand—into a new condition of stable equilibrium. This means sober, industrious, healthy, educated, courageous individuals, in short, good citizens. It means, also, broad-minded, far-sighted, unoppressive industry, good business. A strong and wise government is necessary. Perfect team work between business, government, and people is called for. Fundamentally, the need is always men.

"God give us men! A time like this demands
Strong minds, great hearts, true faith, and ready hands;
Men whom the lust of office does not kill;
Men whom the spoils of office cannot buy;
Men who possess opinions and a will;
Men who have honor, men who will not lie;
Men who can stand before a demagogue,
And damn his treacherous flatteries without winking!
Tall men, sun-crowned, who live above the fog
In public duty and in private thinking:
For while the rabble, with their thumb-worn creeds,
Their large professions and their little deeds,
Mingle in selfish strife,—lo! Freedom weeps,
Wrong rules the land, and waiting Justice sleeps." *

* "Wanted," by Dr. J. G. Holland.

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Paper No. 1283

A RATIONAL FORMULA FOR ASPHALT STREET SURFACES.

By J. Alden Griffin, Assoc. M. Am. Soc. C. E.

Every now and then the question is raised: "What is the proper crown to give an asphalt street?" and there is a discussion as to which of the many formulas of to-day gives the best results.

Having been asked this question many times in the past few years, and especially while connected with municipal improvements in Los Angeles, Cal., the writer has given the matter careful investigation, and, by a comparison of the surfaces proposed by the various formulas, has arrived at the conclusion that the crown rise should vary with the cross-fall as well as the grade of the roadway, and that a crown considerably lower than that proposed by the well-known formula of the late Andrew Rosewater, M. Am. Soc. C. E., should be used on streets having a cross-fall between the gutter grades. The writer even favors one which is slightly lower, where there is no cross-fall in the roadway; and, having reached these conclusions, he proceeded to determine the proper amount of reduction to make in the crown for varying cross-falls. After a great many experiments he adopted the following modification of Mr. Rosewater's formula. This gives the best results, using one-eighth of the cross-fall plus ¾ in. as the reduction factor, but some may wish to change $0.12H$ in the formula to $0.10H$, or even $0.08H$, in order not to reduce the crown quite so much; however, the following is recommended:

$$C = \frac{W\,(100 - 4\,p)}{5\,000} - (0.12H + 0.06)$$

in which W = the width of the roadway between curbs, in feet; p = the percentage of grade longitudinally on the street; H = the cross-fall of the street, or the difference of elevation between the high and low gutters, in feet; and C = the height of the crown above the mean gutter grade, in feet.

TABLE 1.

Width of roadway, in feet.	Percentage of grade on street.	Crown, with no cross-fall.	Crown Rise, for Variable Cross-Falls, in Feet.											
			0.25	0.50	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00
28	1	0.48	0.45	0.42	0.39	0.36	0.33	0.30	....	....	....	....	....	....
28	2	0.45	0.42	0.39	0.36	0.33	0.30	0.27	....	....	....	....	....	....
28	3	0.43	0.40	0.37	0.34	0.31	0.28	0.25	....	....	....	....	....	....
28	4	0.41	0.38	0.35	0.32	0.29	0.26	0.23	....	....	....	....	....	....
28	5	0.39	0.36	0.33	0.30	0.27	0.24	0.21	....	....	....	....	....	....
34	1	0.59	0.56	0.53	0.50	0.47	0.44	0.41	0.38	....	....	...	....	....
34	2	0.57	0.54	0.51	0.48	0.45	0.42	0.39	0.36	....	....	....	....	....
34	3	0.54	0.51	0.48	0.45	0.42	0.39	0.36	0.33	....	....	....	....	...
34	4	0.51	0.48	0.45	0.42	0.39	0.36	0.33	0.30	....	....	....	....	....
34	5	0.48	0.45	0.42	0.39	0.36	0.33	0.30	0.27	....	....	....	....	....
40	1	0.71	0.68	0.65	0.62	0.59	0.56	0.53	0.50	0.47	....	....	....	....
40	2	0.68	0.65	0.62	0.59	0.56	0.53	0.50	0.47	0.44	....	....	....	....
40	3	0.64	0.61	0.58	0.55	0.52	0.49	0.46	0.43	0.40	....	....	....	....
40	4	0.61	0.58	0.55	0.52	0.49	0.46	0.43	0.40	0.37	....	....	....	....
40	5	0.58	0.55	0.52	0.49	0.46	0.43	0.40	0.37	0.34	....	....	....	....
46	1	0.82	0.79	0.76	0.73	0.70	0.67	0.64	0.61	0.58	0.55	....	....	....
46	2	0.79	0.76	0.73	0.70	0.67	0.64	0.61	0.58	0.55	0.52	....	....	....
46	3	0.75	0.72	0.69	0.66	0.63	0.60	0.57	0.54	0.51	0.48	....	....	....
46	4	0.71	0.68	0.65	0.62	0.59	0.56	0.53	0.50	0.47	0.44	....	....	....
46	5	0.68	0.65	0.62	0.59	0.56	0.53	0.50	0.47	0.44	0.41	....	....	....
56	1	1.01	0.98	0.95	0.92	0.89	0.86	0.83	0.80	0.77	0.74	0.71	....	....
56	2	0.97	0.94	0.91	0.88	0.85	0.82	0.79	0.76	0.73	0.70	0.67	....	....
56	3	0.93	0.90	0.87	0.84	0.81	0.78	0.75	0.72	0.69	0.66	0.63	....	....
56	4	0.88	0.85	0.82	0.79	0.76	0.73	0.70	0.67	0.64	0.61	0.58	....	....
56	5	0.84	0.81	0.78	0.75	0.72	0.69	0.66	0.63	0.60	0.57	0.54	....	....
62	1	1.13	1.10	1.07	1.04	1.01	0.98	0.95	0.92	0.89	0.86	0.83	0.80	....
62	2	1.08	1.05	1.02	0.99	0.96	0.93	0.90	0.87	0.84	0.81	0.78	0.75	....
62	3	1.03	1.00	0.97	0.94	0.91	0.88	0.85	0.82	0.79	0.76	0.73	0.70	....
62	4	0.98	0.95	0.92	0.89	0.86	0.83	0.80	0.77	0.74	0.71	0.68	0.65	....
62	5	0.93	0.90	0.87	0.84	0.81	0.78	0.75	0.72	0.69	0.66	0.63	0.60	....
72	1	1.32	1.29	1.26	1.23	1.20	1.17	1.14	1.11	1.08	1.05	1.02	0.99	0.96
72	2	1.27	1.24	1.21	1.18	1.15	1.12	1.09	1.06	1.03	1.00	0.97	0.94	0.91
72	3	1.21	1.18	1.15	1.12	1.09	1.06	1.03	1.00	0.97	0.94	0.91	0.88	0.85
72	4	1.15	1.12	1.09	1.06	1.03	1.00	0.97	0.94	0.91	0.88	0.85	0.82	0.79
72	5	1.09	1.06	1.03	1.00	0.97	0.94	0.91	0.88	0.85	0.82	0.79	0.76	0.73

It will be noticed that on a roadway having a very steep cross-fall the upper gutter will not hold water, which, in the majority of such extreme cases, will do no harm, and will very often save a cross-gutter at the intersection; however, it may be desired at some time to hold the water in the upper gutter, and this may be accomplished, without increasing the side slope of the surface, by shifting the crown to the

upper side of the center of the roadway as shown in Fig. 4, which indicates a special cross-section at that point.

Table 1 is compiled from the formula for the more common roadway widths in Los Angeles.

Figs. 1 to 4 illustrate the comparison between Mr. Rosewater's formula and the modification herein proposed on a 40-ft. roadway

FIG. 1

FIG. 2.

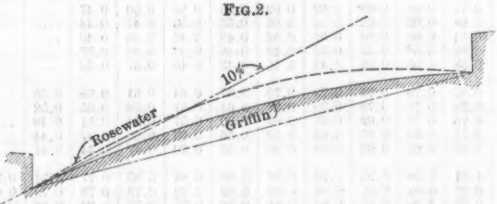


FIG. 3.

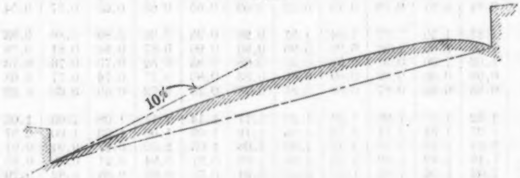


FIG. 4.

having a 1% grade. Fig. 1 is for no cross-fall; Fig. 2 for a cross-fall of 1 ft. between the mean gutters, and Fig. 3 is an extreme case with a cross-fall of 2 ft. between the mean gutters. Fig. 4 illustrates a section in which the lower half meets the formula and the upper half is modified to hold the water in the upper gutter.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1284

THE STORAGE OF FLOOD-WATERS FOR IRRIGATION: A STUDY OF THE SUPPLY AVAILABLE FROM SOUTHERN CALIFORNIA STREAMS.*

By A. M. Strong, Assoc. M. Am. Soc. C. E.

With Discussion by Charles H. Lee, Assoc. M. Am. Soc. C. E.

Many of the richest sections of California are valleys and plains at the foot of abrupt mountain ranges. The climate is suitable for raising citrus and semi-tropical fruits, but, due to the long summer droughts, this can only be done where there is sufficient water for irrigation. The value of the products has been great, and, beginning with the small irrigation ditches of the Spanish days, the most perfect systems of irrigation in the United States have been built up.

The source of all the water used in these sections is in the streams coming from the neighboring mountain ranges. It is secured by direct diversion, by storage in surface reservoirs, or by pumping from the underground gravel beds fed by these streams. A large part of the total run-off of the streams comes during a short rainy season, or, in the case of those heading in the highest ranges, with the first hot weather of the summer. Rainfall records for 34 years in Los Angeles show that an average of 75% fell during December, January, February, and March, and the run-off of Southern California streams shows a corresponding percentage. In the case of some of the streams head-

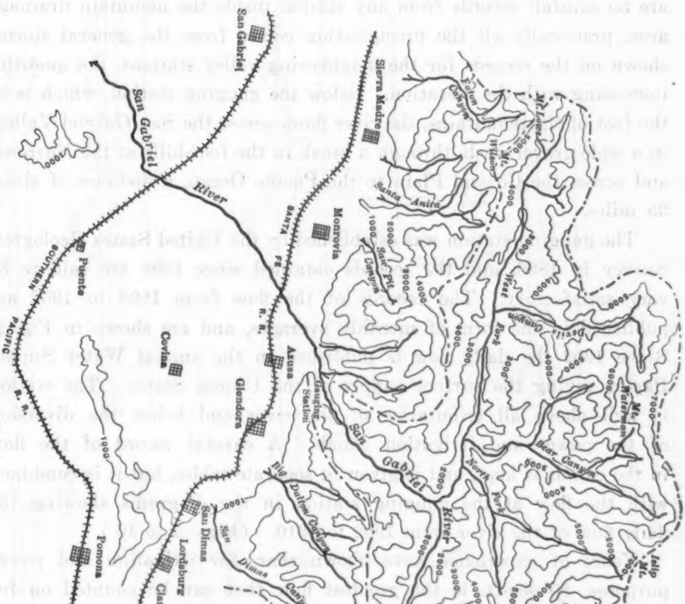
* Presented at the meeting of September 3d, 1913.

ing in the snow fields of the Sierra Nevadas, from 12 to 15% of the total run-off comes during a 10-day period in June or July. In the middle and late summer months, when irrigation is needed most, the streams have reached their low-water flow. For this reason many large storage reservoirs have been built, and large areas are irrigated by pumping from underground basins.

As a rule, the area of land which can be placed under cultivation is measured by the quantity of water available, and great expense is warranted in increasing the supply. In many places there are no practical reservoir sites directly on the streams, and in others the available reservoir capacity is only a small part of the total run-off. Even where conditions are most favorable for storage, there are many seasons when it is insufficient for the total flood run-off. Irrigation has now reached a point where all the flow available for direct diversion is in use, and, particularly in Southern California, reservoirs have been constructed on all available sites on the streams. In many places the pumping draft from underground basins is as great as the inflow, or greater. The percentage of the total run-off which can be put to beneficial use can be increased only by diverting the flood flow to reservoirs outside of the natural drainage, or by increasing the flow into the underground basins.

The question of the quantity of water available for storage in this way is becoming very important wherever the water supply for irrigation is extensively developed. The principal problem is the diversion of a large flow from the flood-water for a short time and its storage for a smaller flow during the entire irrigation season. This is now being done in a number of the late projects in various parts of the West. The object of the investigation described in this paper was to get some idea as to the quantity of water now being wasted, but still available for storage in this way. The San Gabriel River, with unusually good records of run-off, was taken as a typical case.

This river drains a large portion of the southern slope of the San Gabriel Range, and is one of the three principal streams of Southern California. The mountain area above the gauging station contains 222 sq. miles, and is somewhat rectangular, as shown on the map, Fig. 1. The elevations range from 1 000 ft. at the gauging station to 10 080 ft. at San Antonio Peak, with an average of about 4 000 ft. To an elevation of about 5 000 ft., the ground is covered with a dense



DRAINAGE AREA
of the
SAN GABRIEL RIVER
Los Angeles County, California

SCALE OF MILES
0 1 2 3 4 5

FIG. 1

growth of brush; above that level there is more or less timber. The geological formation is granite, with a thin soil covering. Precipitation is in the form of rain, except on the higher peaks. Though there are no rainfall records from any station inside the mountain drainage area, practically all the precipitation comes from the general storms shown on the records for the neighboring valley stations, the quantity increasing with the elevation. Below the gauging station, which is at the foot of the mountains, the river flows across the San Gabriel Valley, in a wide gravel wash, through a break in the foot-hills at the Narrows, and across the Coastal Plain to the Pacific Ocean, a distance of about 35 miles.

The gauging station was established by the United States Geological Survey in 1896, and the records obtained since 1898 are said to be very satisfactory. The records of the flow from 1896 to 1902 are published in the form of monthly averages, and are shown in Fig. 2. Since 1902 the daily flow is published in the annual Water Supply Papers giving the surface supply of the United States. The station is well above all tributaries of the river and below the diversion of the power and irrigation canals. A careful record of the flow in the canals is kept, and is given in separate tables, but it is combined with the flow at the gauging station in the diagrams showing the daily flow of the river from 1902 to 1910. (Figs. 3 to 10.)

Years of experience have shown that, for irrigation and power purposes, 80 sec-ft. is the greatest flow that can be counted on for any considerable length of time, and, as a result, this has been the capacity of the diversion canals. Except during the rainy season, this diversion capacity is as great as the total flow of the stream, or greater. Fig. 11 shows the number of days, during each of the eight years, in which the flow exceeded 80 sec-ft., and the duration of the varying flows up to 1 000 sec-ft. The average for those years for which the flow was more than 80 sec-ft. is 148, and that in which it was more than 1 000 sec-ft. is 13½ days. The individual years differ greatly from this average.

No place has ever been found on the drainage area where storage could be obtained, and all the flow in excess of 80 sec-ft. runs down the gravel wash in the San Gabriel Valley. Here a certain quantity passes into the underground storage in the gravel beds, and supplies the numerous pumping plants scattered over the surface of the under-

MEAN, MAXIMUM, AND MINIMUM FLOW,
SAN GABRIEL RIVER
BY MONTHS, 1896 TO 1901-02.

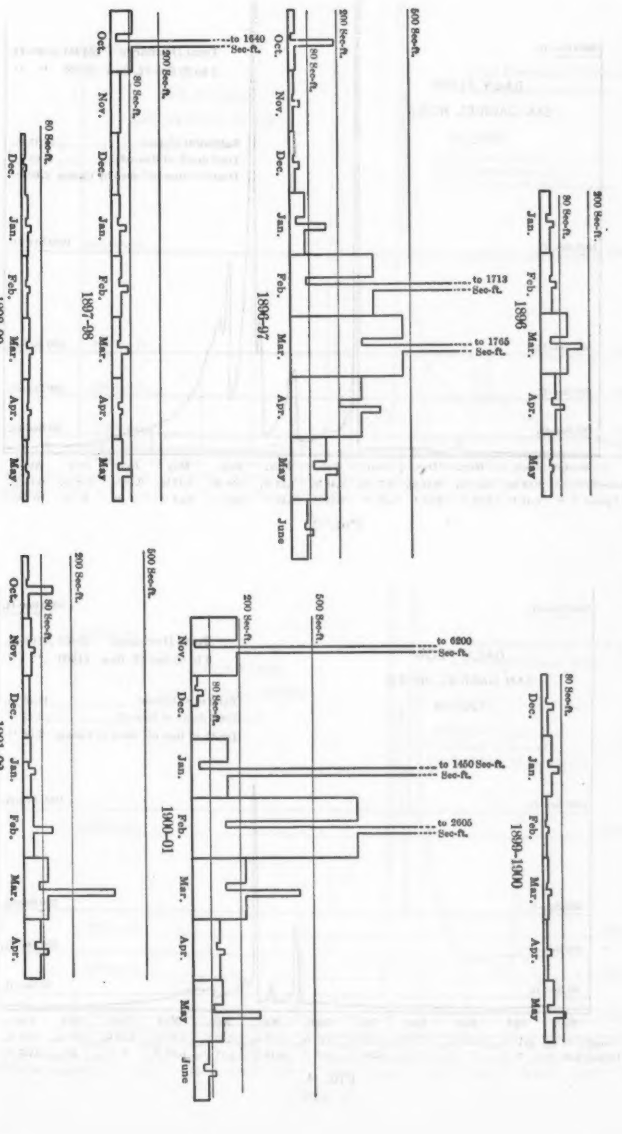


FIG. 2.

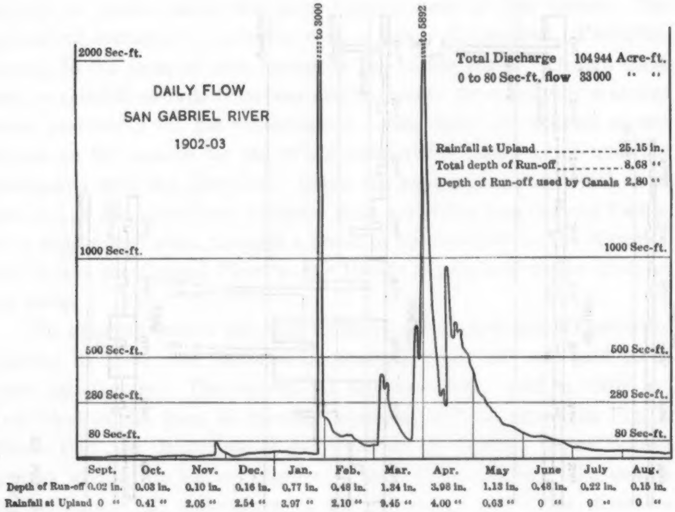

DAILY FLOW
SAN GABRIEL RIVER
1902-03
to 3000
to 5892
Total Discharge 104944 Acre-ft.
0 to 80 Sec-ft. flow 33000 " "
Rainfall at Upland 25.15 in.
Total depth of Run-off 8.68 "
Depth of Run-off used by Canals 2.80 "
2000 Sec-ft.
1000 Sec-ft.
500 Sec-ft.
280 Sec-ft.
80 Sec-ft.
Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug.
Depth of Run-off 0.02 in. 0.03 in. 0.10 in. 0.16 in. 0.77 in. 0.48 in. 1.84 in. 3.98 in. 1.13 in. 0.48 in. 0.22 in. 0.15 in.
Rainfall at Upland 0 " 0.41 " 2.05 " 2.54 " 3.97 " 2.10 " 9.45 " 4.00 " 0.63 " 0 " 0 " 0 "

FIG. 3.

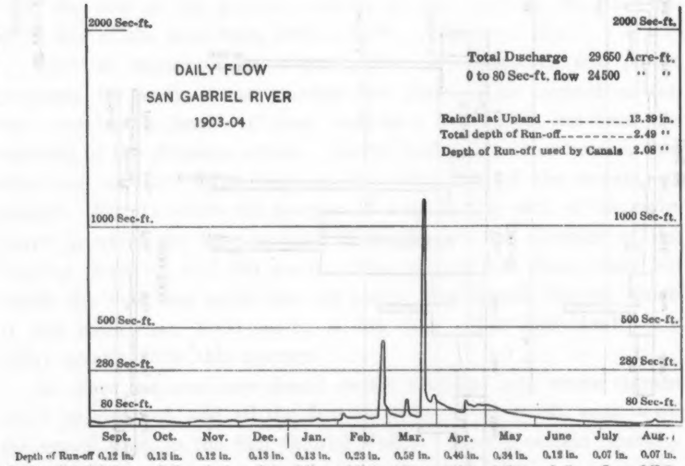

DAILY FLOW
SAN GABRIEL RIVER
1903-04
Total Discharge 29650 Acre-ft.
0 to 80 Sec-ft. flow 24500 " "
Rainfall at Upland 13.39 in.
Total depth of Run-off 2.49 "
Depth of Run-off used by Canals 2.08 "
2000 Sec-ft.
1000 Sec-ft.
500 Sec-ft.
280 Sec-ft.
80 Sec-ft.
Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug.
Depth of Run-off 0.12 in. 0.13 in. 0.12 in. 0.13 in. 0.13 in. 0.23 in. 0.58 in. 0.46 in. 0.34 in. 0.12 in. 0.07 in. 0.07 in.
Rainfall at Upland 0.42 " 0 " 0 " 0 " 0.39 " 4.26 " 6.18 " 1.77 " 0.30 " 0 " Tr. 0.07 "

FIG. 4.

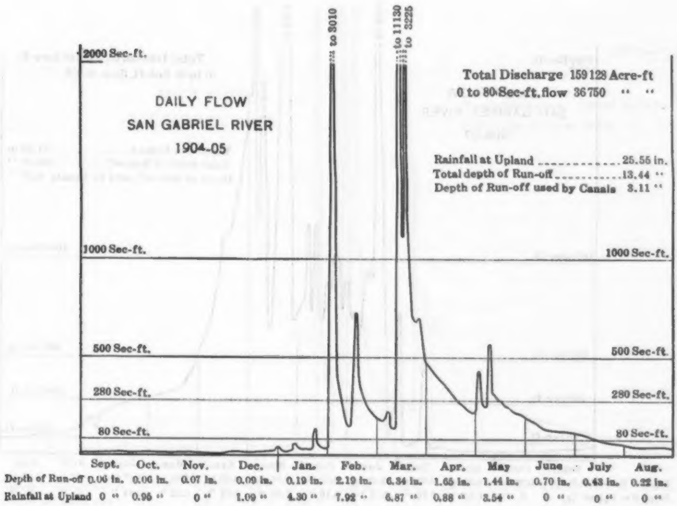

DAILY FLOW
SAN GABRIEL RIVER
1904-05
to 3010
to 11130
to 3225
Total Discharge 159128 Acre-ft
0 to 80 Sec-ft. flow 36750 " "
Rainfall at Upland 25.55 in.
Total depth of Run-off 13.44 "
Depth of Run-off used by Canals 3.11 "
2000 Sec-ft.
1000 Sec-ft.
500 Sec-ft.
280 Sec-ft.
80 Sec-ft.
Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug.
Depth of Run-off 0.06 in. 0.06 in. 0.07 in. 0.09 in. 0.19 in. 2.19 in. 6.34 in. 1.65 in. 1.44 in. 0.70 in. 0.43 in. 0.22 in.
Rainfall at Upland 0 " 0.95 " 0 " 1.09 " 4.30 " 7.92 " 6.87 " 0.88 " 3.54 " 0 " 0 " 0 "

FIG. 5.

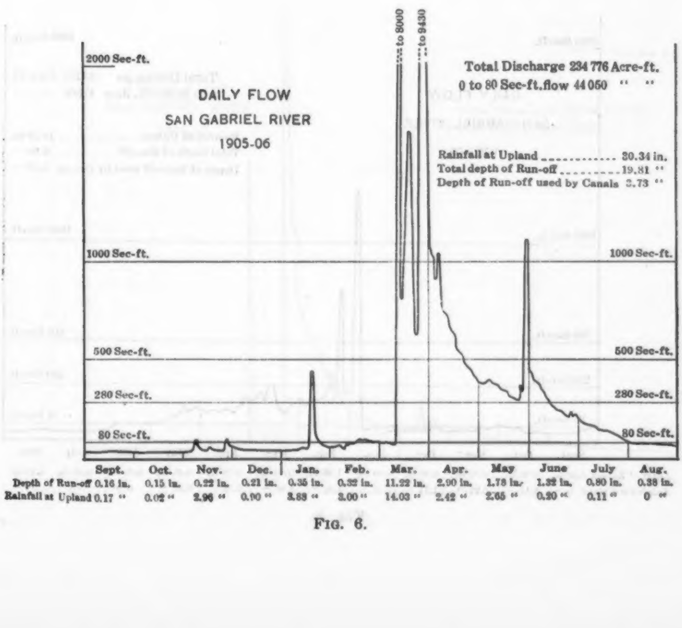

DAILY FLOW
SAN GABRIEL RIVER
1905-06
to 8000
to 9430
Total Discharge 234776 Acre-ft.
0 to 80 Sec-ft. flow 44050 " "
Rainfall at Upland 30.34 in.
Total depth of Run-off 19.81 "
Depth of Run-off used by Canals 2.73 "
2000 Sec-ft.
1000 Sec-ft.
500 Sec-ft.
280 Sec-ft.
80 Sec-ft.
Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug.
Depth of Run-off 0.16 in. 0.15 in. 0.22 in. 0.21 in. 0.35 in. 0.32 in. 11.22 in. 2.90 in. 1.78 in. 1.32 in. 0.80 in. 0.38 in.
Rainfall at Upland 0.17 " 0.02 " 2.96 " 0.00 " 3.88 " 3.00 " 14.03 " 2.42 " 2.65 " 0.20 " 0.11 " 0 "

FIG. 6.

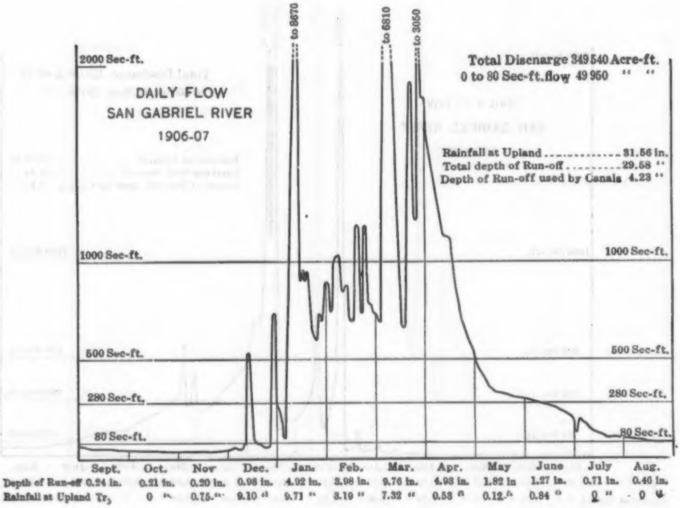

to 8670
to 6810
to 3050
2000 Sec-ft.
DAILY FLOW
SAN GABRIEL RIVER
1906-07
Total Discharge 349 540 Acre-ft.
0 to 80 Sec-ft. flow 49 950 " "
Rainfall at Upland .... 31.56 in.
Total depth of Run-off .... 29.58 "
Depth of Run-off used by Canals 4.23 "
1000 Sec-ft.
500 Sec-ft.
280 Sec-ft.
80 Sec-ft.
Sept. Oct. Nov Dec. Jan. Feb. Mar. Apr. May June July Aug.
Depth of Run-off 0.24 in. 0.21 in. 0.20 in. 0.98 in. 4.92 in. 3.98 in. 9.76 in. 4.93 in. 1.82 in 1.27 in. 0.71 in. 0.46 in.
Rainfall at Upland Tr. 0 " 0.75 " 9.10 " 9.71 " 3.19 " 7.32 " 0.53 " 0.12 " 0.84 " 0 " 0 "

FIG. 7.

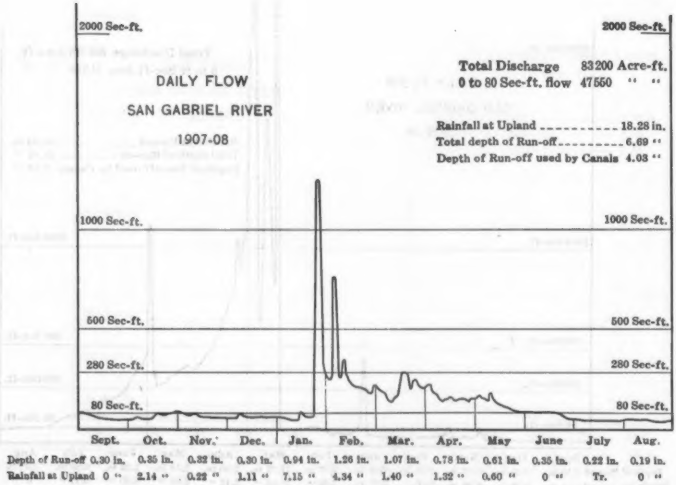
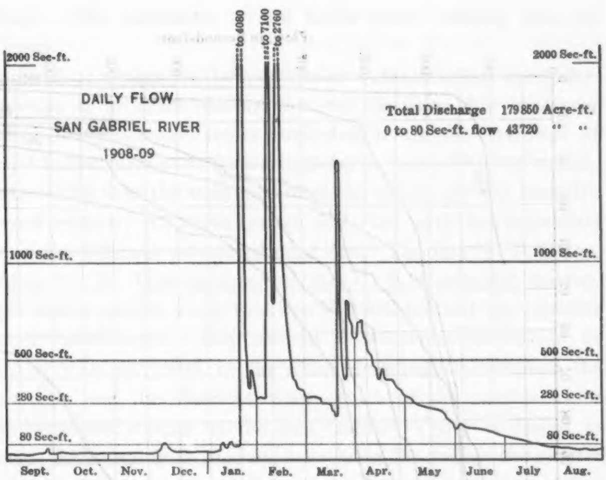
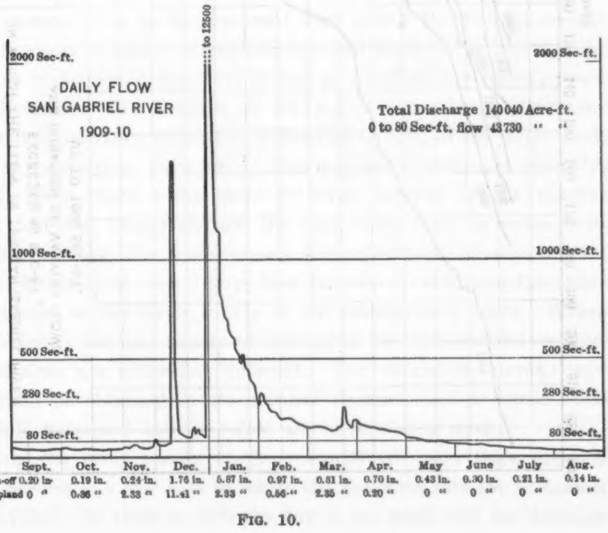

2000 Sec-ft.
2000 Sec-ft.
DAILY FLOW
SAN GABRIEL RIVER
1907-08
Total Discharge 83 200 Acre-ft.
0 to 80 Sec-ft. flow 47 550 " "
Rainfall at Upland .... 18.28 in.
Total depth of Run-off .... 6.69 "
Depth of Run-off used by Canals 4.03 "
1000 Sec-ft.
500 Sec-ft.
280 Sec-ft.
80 Sec-ft.
Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug.
Depth of Run-off 0.30 in. 0.35 in. 0.32 in. 0.30 in. 0.94 in. 1.26 in. 1.07 in. 0.78 in. 0.61 in. 0.35 in. 0.22 in. 0.19 in.
Rainfall at Upland 0 " 2.14 " 0.22 " 1.11 " 7.15 " 4.34 " 1.40 " 1.32 " 0.60 " 0 " Tr. 0 "

FIG. 8.



	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.
Depth of Run-off	0.15 in.	0.17 in.	0.17 in.	0.24 in.	2.14 in.	5.02 in.	2.22 in.	2.36 in.	1.29 in.	0.72 in.	0.43 in.	0.26 in.
Rainfall at Upland	1.86 "	1.03 "	0.36 "	1.27 "	11.08 "	7.04 "	3.95 "	0.17 "	0 "	0 "	0.05 "	0.02 "

FIG. 9.



	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.
Depth of Run-off	0.20 in.	0.19 in.	0.24 in.	1.76 in.	5.87 in.	0.97 in.	0.81 in.	0.70 in.	0.43 in.	0.30 in.	0.21 in.	0.14 in.
Rainfall at Upland	0 "	0.36 "	2.33 "	11.41 "	2.33 "	0.56 "	2.35 "	0.20 "	0 "	0 "	0 "	0 "

FIG. 10.

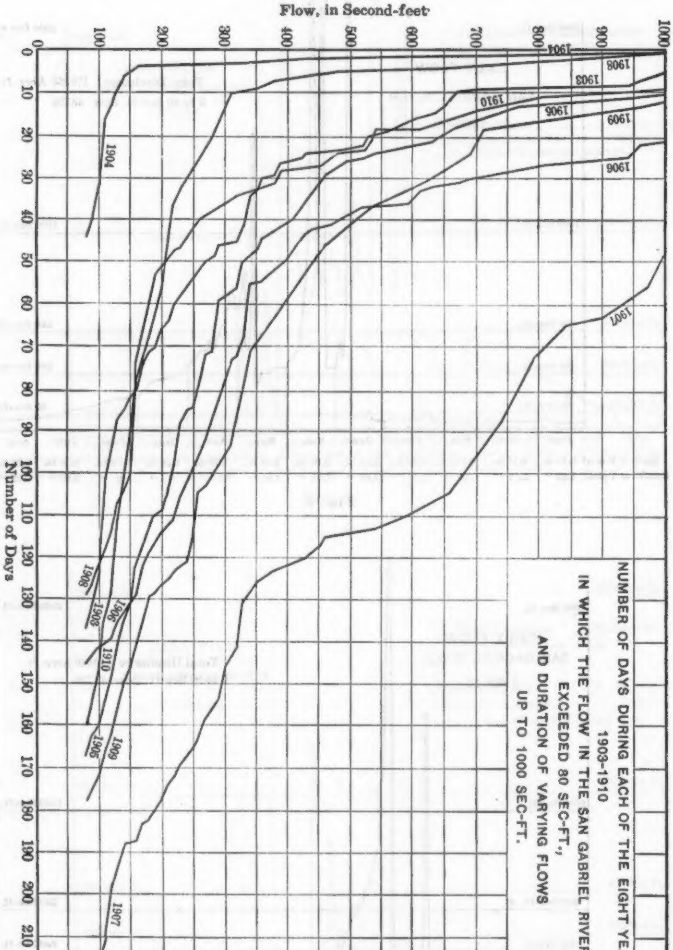

Flow, in Second-feet
0
100
200
300
400
500
600
700
800
900
1000
Number of Days
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
210
220
230
1904
1908
1903
1910
1906
1909
1905
1907
NUMBER OF DAYS DURING EACH OF THE EIGHT YEARS,
1903-1910
IN WHICH THE FLOW IN THE SAN GABRIEL RIVER
EXCEEDED 80 SEC-FT.,
AND DURATION OF VARYING FLOWS
UP TO 1000 SEC-FT.

FIG. 11.

ground basin. The remainder is now waste water running into the Pacific Ocean.

It is difficult to determine the quantity of water entering the underground storage, as no exact measurements of the waste flow have ever been made. The principal factors governing it are the character of the material in the surface of the wash, the area covered by the stream, the velocity of the flow, the temperature of the water, and the quantity of suspended matter. All these factors will vary with the individual character of the different periods of flood flow. The late W. B. Clapp, M. Am. Soc. C. E., Hydrographer of the U. S. Geological Survey, made some measurements along this line in 1903, but did not consider them at all satisfactory. They showed a maximum absorption of 86 sec-ft., on May 23d, 1903, in the main river channel between the gauging station and the Narrows, where much of the surplus water of the underground storage of the San Gabriel Valley is forced to the surface. As pumping is used extensively in the valley, the greater part of the natural underground flow is now being put to use, and it must be taken into consideration in calculating the available supply for further economic use.

In the case of flood-water diverted from the natural channel for surface storage, it is to be presumed that it will be used for irrigation on territory tributary to the underground basin fed by the stream. All return water would flow into it and go to offset any losses caused by the diversion. The quantity of this return water is not determinable, but it is variously estimated at from 10 to 50% of the water used. Again, any diversion, even up to the maximum economic quantity, will still leave times when there is water flowing in the natural channel, and will simply shorten the time when there is water flowing in that channel. For these reasons it is not considered necessary for any such diversion to pass a large flow in order to maintain the existing conditions of the water supply of the underground basin. Where the diversion is for the purpose of increasing the underground storage, the conditions are somewhat different. The maximum storage possible should be obtained in the natural channel, and a considerable flow should be passed into it before any diversion is made.

During the rainy season of the majority of years there are one or more periods of a few days in which the flow runs into the thousands of second-feet. In these periods the flow is too great and the duration

too short to warrant handling by any diversion. That part of the total run-off of the San Gabriel River which is available for further economic use by storage, either surface or underground, is the flow in excess of 80 sec-ft. plus a reasonable allowance for water entering the underground basin, but not including the extreme floods.

For the purpose of determining this flow, there are available the records of the daily flow for the eight years, as shown on the hydrographs. The only way to test the reliability of any deductions drawn from these records, in relation to long periods of time, is by a comparison with the rainfall records which have been kept for a much longer time. This comparison will only apply to general conditions, for, though the stream run-off is dependent directly on the precipitation, the percentage showing as surface run-off varies greatly. Much depends on the general seasonal conditions and on the character of the individual storms. Fig. 12 illustrates some of the characteristics of the rainfall and run-off during these 8 years. It shows how greatly the percentage of the run-off depends on the conditions under which the rainfall comes. In the season of 1905-06 a few very heavy storms gave a large percentage of run-off, of which a great part was in extreme floods, while nearly the same rainfall in 1908-09, coming in a large number of smaller storms, gave a smaller total run-off with a larger quantity under moderate flood stages. Similarly, the season of 1904-05, with the precipitation falling on a dry water-shed, gave less run-off for all stages of flow. These are only a few of the factors which govern the quantity of run-off resulting from the precipitation of any given season. However, the general seasonal conditions of precipitation give some valuable information in regard to the possible supply for storage.

Fig. 13 is a diagram showing the rainfall for a number of Southern California points. Fig. 14 shows the seasonal rainfall of a number of places near the drainage area, expressed as percentages of excess or deficiency from the long-time mean. These diagrams show that the period from 1897 to 1900 was the dryest in 40 years, or the dryest in 60 years, if the San Diego records are of any value for comparison. This is as far back as the records extend, but early California history indicates that there has been one other period of that kind within the past 100 years. This period might be taken into consideration as a limiting condition, but any short-time average, using the records

during that time, would give results which would be too low. The average rainfall from 1902 to 1910 is very near the long-time average, and the different seasons represent all but very unusual conditions. The season of 1903-04 had a very light rainfall and that of 1906-07 an unusually heavy one. Any conclusions drawn from the records of these eight seasons, except in abnormal cases, should cover the conditions to be met in the future.

AVERAGE RAINFALL FOR 7 FOOT-HILL STATIONS AND
DEPTH OF RUN-OFF FOR DIFFERENT DIVISIONS OF THE FLOW
SAN GABRIEL RIVER

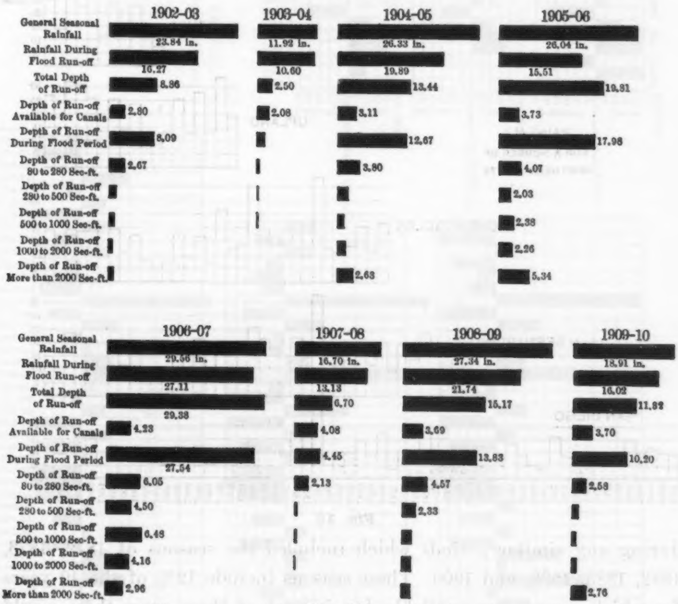


FIG. 12.

In calculating the available supply for surface storage, it is at once evident from the rainfall records that there are a large number of alternating seasons in which the run-off is comparatively small. Any large surface reservoir drawing its supply from a diversion of the flood flow would have to furnish the supply for more than one irrigation with each filling. In order to calculate the available supply for such a reservoir, it is necessary to select a period during which the run-off

represents a limiting minimum condition. Figs. 15 to 18 show the total discharge for the flood periods under consideration and the discharge which would have been available for varying diversion capacities. Fig. 19 shows the discharge curve for all flows up to 1 000 sec-ft. for the several seasons. During these eight years, the period from January, 1903, to October, 1904, gives the limiting minimum condition. The rainfall records indicate that, other things being equal, the supply available during this period, if used during two irrigation seasons, would have been greater than that which would have been available

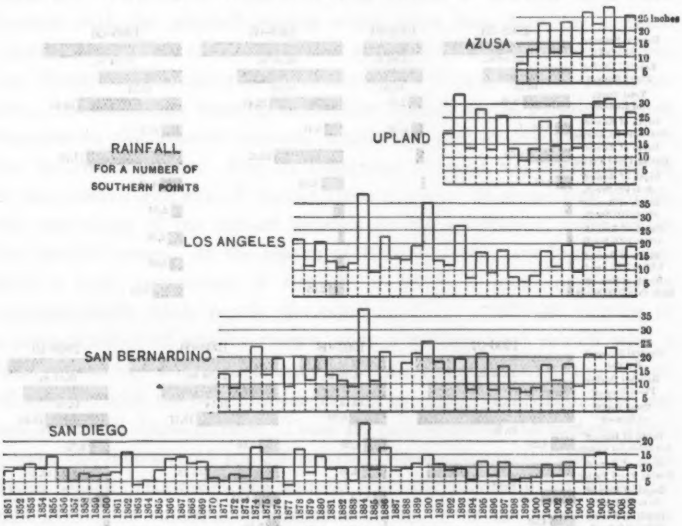


FIG. 13.

during any similar periods which included the seasons of 1872, 1873, 1882, 1883, 1899, and 1900. These seasons include 12% of the 40 years for which records are available, but in each of these years there would have been some storage water, in some of them probably enough for the half-reservoir capacity necessary for one irrigation season.

It was found, during the drought from 1897 to 1900, that practically every irrigation system in Southern California depending on a surface water supply was covering too great a territory for such a period. It was also shown that orchards could be kept alive for one or two years with very little water, the only loss being the crops for those seasons.

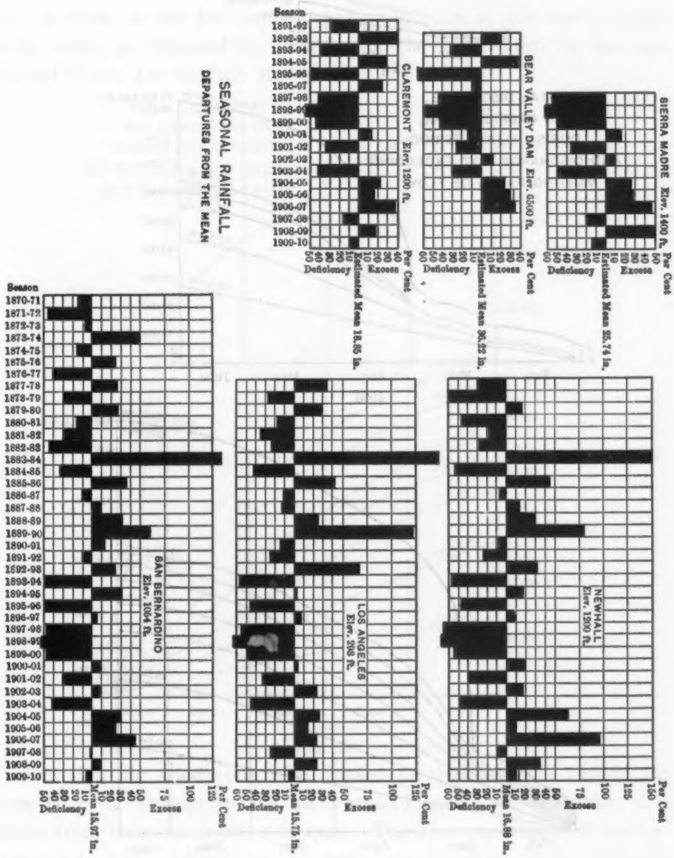

SEASONAL RAINFALL
DEPARTURES FROM THE MEAN
SIERRA MADRE Elev. 1400 ft.
Estimated Mean 25.74 in.
BEAR VALLEY DAM Elev. 6500 ft.
Estimated Mean 36.22 in.
CLAREMONT Elev. 1200 ft.
Estimated Mean 18.85 in.
SAN BERNARDINO Elev. 1054 ft.
Mean 15.07 in.
LOS ANGELES Elev. 293 ft.
Mean 15.75 in.
NEWHALL Elev. 1200 ft.
Mean 16.88 in.
Season
Deficiency
Excess
Per Cent

FIG. 14.

The increased cost of water for irrigation, if it should be considered necessary to hold enough in storage to cover such a period, would be much greater than the value of the lost crops. Also, the area of land

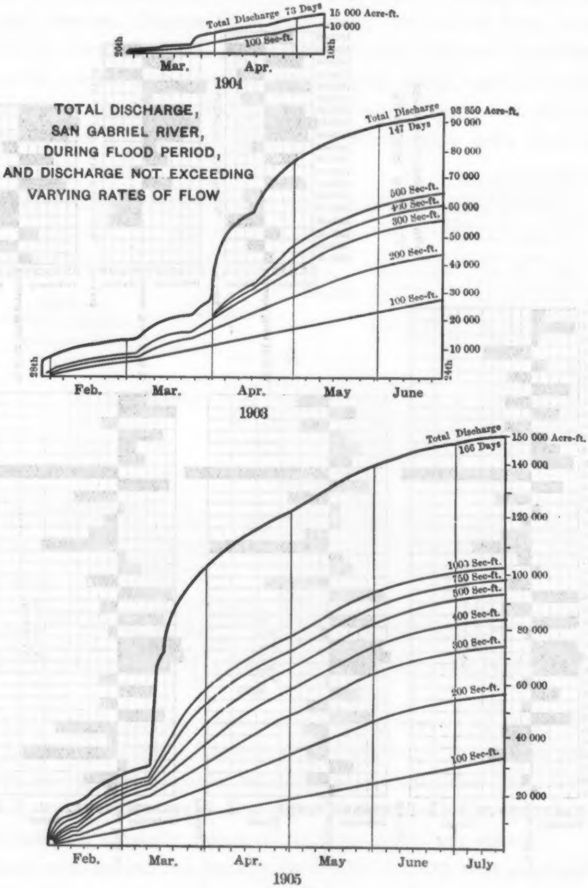


FIG. 15.

which could be placed under cultivation with the existing supply, and the resulting value of that supply, would have to be greatly reduced. Another point that must be taken into consideration is that there can

be a resort to pumping, even if at great cost, to cover short periods of deficient supply. For these reasons it is not thought necessary to consider extreme conditions for the economic minimum supply, and it is thought that the period from January, 1903, to October, 1904, may be taken as the limiting economic condition of the water supply which could be obtained by diversion of the flood-waters of the San Gabriel River for surface storage.

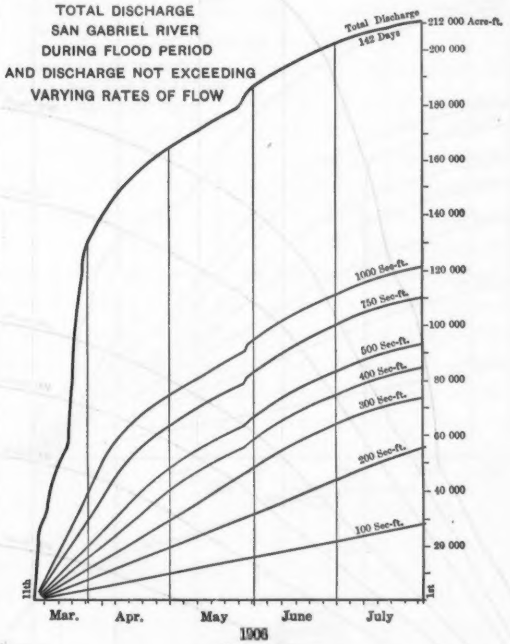


FIG. 16.

The limiting conditions with regard to the available supply for increasing the storage in the underground basins are somewhat different from those for surface storage. The slow movement of the underground waters tends to equalize the varying flows of the different years, and the great extent of the underground basins gives almost unlimited storage capacity. Where the present pumping draft is as great as the natural supply, or greater, the full capacity of these basins, above the limit of economic pumping, is available for in-

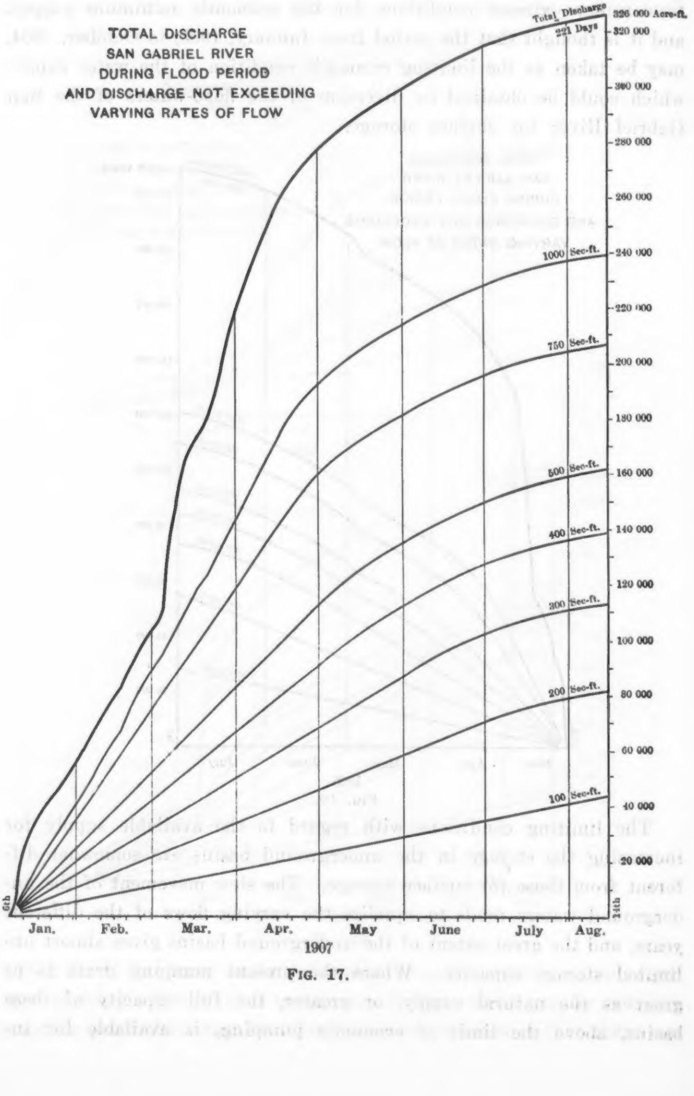

TOTAL DISCHARGE
SAN GABRIEL RIVER
DURING FLOOD PERIOD
AND DISCHARGE NOT EXCEEDING
VARYING RATES OF FLOW
Total Discharge
221 Days
1000 Sec-ft.
750 Sec-ft.
500 Sec-ft.
400 Sec-ft.
300 Sec-ft.
200 Sec-ft.
100 Sec-ft.
326 000 Acre-ft.
320 000
300 000
280 000
260 000
240 000
220 000
200 000
180 000
160 000
140 000
120 000
100 000
80 000
60 000
40 000
20 000
6th
14th
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
1907

FIG. 17.

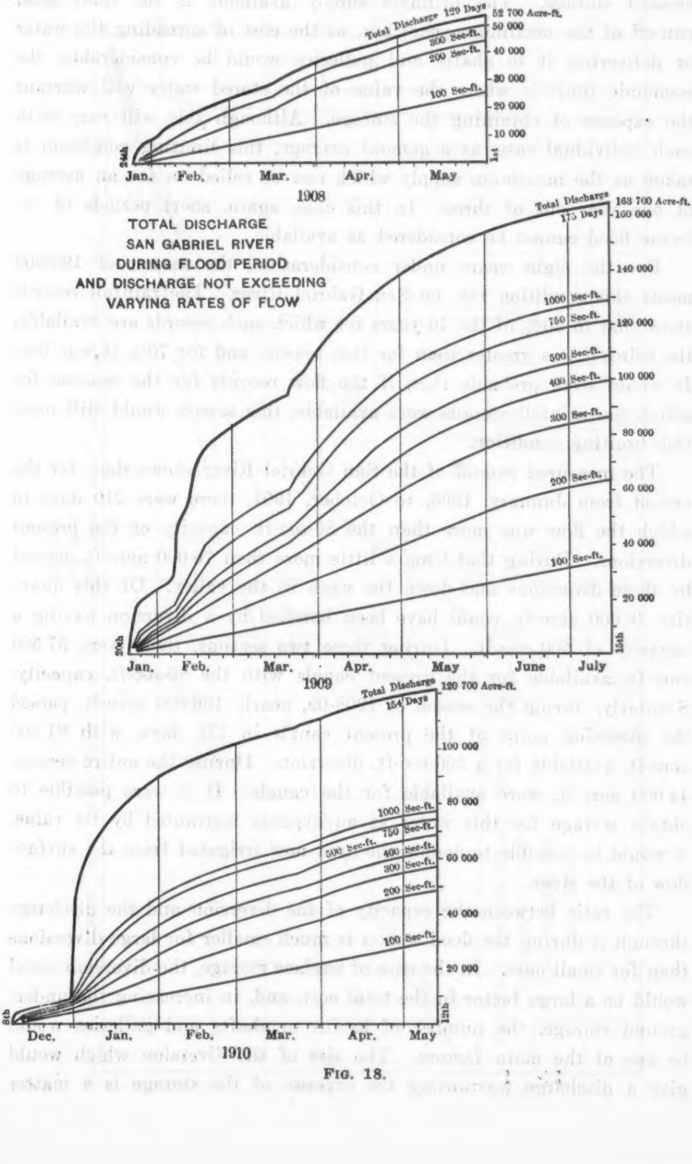

Total Discharge 120 Days
52 700 Acre-ft.
300 Sec-ft.
200 Sec-ft.
100 Sec-ft.
50 000
40 000
30 000
20 000
10 000
Jan.
Feb.
Mar.
Apr.
May
1908
TOTAL DISCHARGE
SAN GABRIEL RIVER
DURING FLOOD PERIOD
AND DISCHARGE NOT EXCEEDING
VARYING RATES OF FLOW
Total Discharge 175 Days
163 700 Acre-ft.
160 000
140 000
1000 Sec-ft.
750 Sec-ft.
120 000
500 Sec-ft.
400 Sec-ft.
100 000
300 Sec-ft.
80 000
200 Sec-ft.
60 000
40 000
100 Sec-ft.
20 000
20th
15th
Jan.
Feb.
Mar.
Apr.
May
June
July
1909
Total Discharge 154 Days
120 700 Acre-ft.
100 000
80 000
1000 Sec-ft.
750 Sec-ft.
500 Sec-ft.
400 Sec-ft.
300 Sec-ft.
60 000
200 Sec-ft.
40 000
100 Sec-ft.
20 000
8th
12th
Dec.
Jan.
Feb.
Mar.
Apr.
May
1910

FIG. 18.

creased storage. The ultimate supply available is the total flood run-off of the maximum year; but, as the cost of spreading the water or delivering it to shafts and galleries would be considerable, the economic limit is where the value of the stored water will warrant the expense of obtaining the storage. Although this will vary with each individual case, as a general average, this limiting condition is taken as the maximum supply which can be relied on for an average of one year out of three. In this case, again, short periods of extreme flood cannot be considered as available.

For the eight years under consideration, the season of 1908-09 meets this condition for the San Gabriel River. The rainfall records show that in 30% of the 40 years for which such records are available, the rainfall was greater than for that season, and for 70% it was less. It would seem probable that, if the flow records for the seasons for which the rainfall records were available, this season would still meet this limiting condition.

The measured run-off of the San Gabriel River shows that, for the period from January, 1903, to October, 1904, there were 210 days in which the flow was more than the 80-sec-ft. capacity of the present diversions. During that time a little more than 76 000 acre-ft. passed by these diversions and down the wash in the valley. Of this quantity 46 000 acre-ft. could have been handled by a diversion having a capacity of 500 sec-ft. During these two seasons, there were 57 500 acre-ft. available for the present canals with the 80-sec-ft. capacity. Similarly, during the season of 1908-09, nearly 136 000 acre-ft. passed the diversion point of the present canals in 175 days, with 81 500 acre-ft. available for a 500-sec-ft. diversion. During the entire season, 44 000 acre-ft. were available for the canals. If it were possible to obtain storage for this water at an expense warranted by its value, it would be possible to double the area now irrigated from the surface flow of the river.

The ratio between the capacity of the diversion and the discharge through it during the flood season is much smaller for large diversions than for small ones. In the case of surface storage, the diversion canal would be a large factor in the total cost, and, in increasing the underground storage, the number of basins or shafts and galleries would be one of the main factors. The size of the diversion which would give a discharge warranting the expense of the storage is a matter

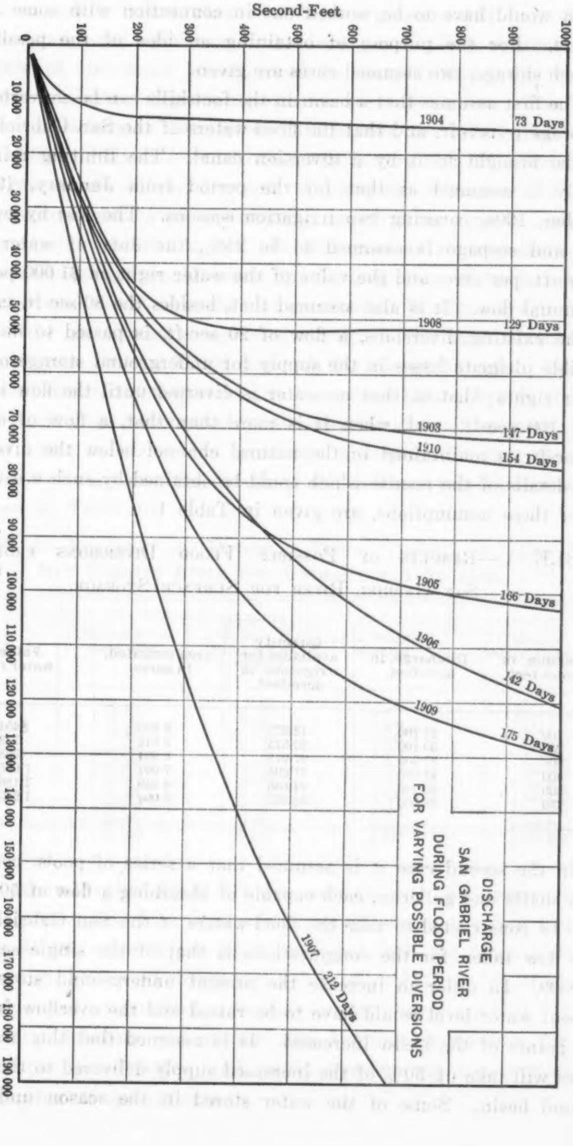

Second-Feet
0
100
200
300
400
500
600
700
800
900
1000
10 000
20 000
30 000
40 000
50 000
60 000
70 000
80 000
90 000
100 000
110 000
120 000
130 000
140 000
150 000
160 000
170 000
180 000
190 000
Acre-Feet
1904
73 Days
1908
129 Days
1903
147 Days
1910
154 Days
1905
166 Days
1906
142 Days
1909
175 Days
1907
212 Days
DISCHARGE
SAN GABRIEL RIVER
DURING FLOOD PERIOD
FOR VARYING POSSIBLE DIVERSIONS

FIG. 19.

which would have to be worked out in connection with some special project. For the purpose of obtaining an idea of the possibilities of such storage, two assumed cases are given.

The first assumes that a basin in the foot-hills can be converted into a storage reservoir, and that the flood-waters of the San Gabriel River can be brought to it by a diversion canal. The limiting minimum supply is assumed as that for the period from January, 1903, to October, 1904, covering two irrigation seasons. The loss by evaporation and seepage is assumed to be 25%, the duty of water to be 2 acre-ft. per acre, and the value of the water right as $1 000 per inch of annual flow. It is also assumed that, besides the 80-sec-ft. capacity of the existing diversions, a flow of 20 sec-ft. is passed to meet any possible ultimate losses in the supply for underground storage or prior water rights; that is, that no water is diverted until the flow is more than 100 sec-ft., and, when it is more than that, a flow of at least 20 sec-ft. is maintained in the natural channel below the diversions. The details of the results which could be obtained by such a diversion, under these assumptions, are given in Table 1.

TABLE 1.—RESULTS OF POSSIBLE FLOOD DIVERSIONS FROM THE SAN GABRIEL RIVER FOR SURFACE STORAGE.

Diversion, in second-feet.	Discharge, in acre-feet.	Quantity available for irrigation, in acre-feet.	Area irrigated, in acres.	Value of water right.
100	17 700	13 275	3 320	$454 840
200	30 100	22 575	5 644	773 223
300	37 300	27 975	6 994	958 178
400	42 260	31 695	7 924	1 085 588
500	46 180	34 635	8 659	1 186 283
750	48 500	36 375	9 094	1 245 878

In the second case it is assumed that a series of pools, connected with shafts and galleries, each capable of absorbing a flow of 50 sec-ft., is to be constructed to take the flood-waters of the San Gabriel River. The flow taken for the computations is that of the single season of 1908-09. In order to increase the present underground storage, the present water level would have to be raised and the overflow from the low points of the basin increased. It is assumed that this and other losses will take up 50% of the increased supply delivered to the underground basin. Some of the water stored in the season under con-

sideration, and in years when the flow is greater, will have to be used to equalize the deficiency of the dryer years. It is assumed that two-thirds of the water which could be stored during this season would be available for pumping during any one season. The duty of water, as before, is assumed to be 2 acre-ft. per acre. The value of the additional water stored in this way is very difficult to estimate. Under general conditions, the cost of pumping is greater than the cost of maintenance of the distributing system from a surface supply, and the capital invested is less. In this case, as an increase in the underground storage would result in the irrigation of lands now of little value, and would make them of great value, it is assumed that the value of the net available increased storage is the same as that for surface storage, that is, $1 000 per inch for the water right. Besides the present diversion of 80 sec-ft., it is assumed that a flow of at least 100 sec-ft. is to be maintained in the natural channel whenever possible, in order to assure the maximum absorption in that channel. With these assumptions, the details of the different possible diversions are given in Table 2.

TABLE 2.—RESULTS OF POSSIBLE FLOOD DIVERSIONS FROM THE SAN GABRIEL RIVER FOR UNDERGROUND STORAGE.

Diversion, in second-feet.	Discharge, in acre-feet.	Quantity available for pumping, in acre-feet.	Area irrigated, in acres.	Value of water right.
50	12 460	4 153	2 077	$284 549
100	23 420	7 806	3 903	434 711
150	31 920	10 640	5 320	728 840
200	38 240	12 747	6 374	873 238
250	43 360	14 453	7 227	990 099
300	47 600	15 867	7 934	1 086 956
350	51 300	17 100	8 550	1 187 350
400	54 500	18 167	9 084	1 244 508
450	57 220	19 073	9 537	1 306 569
500	59 620	19 873	9 937	1 361 569

The general value of $1 000 per miner's inch for water rights scarcely gives a fair idea of the local conditions in the San Gabriel Valley. At the current rate of 2 cents per inch per hour delivered at the groves, the annual charge per inch is about $175, or 7% of $2 500. This, calculated in acre-feet, gives a warranted investment of $171.23 per acre-ft., including both storage and distributing systems, for the net available stored water, and would warrant the construction of very extensive storage systems.

There are many small streams in Southern California having a run-off which is similar to that of the San Gabriel River, and the only way to get any idea of the available supply for storage from them is by comparison with it. The rainfall on their drainage areas is about the same. It is probable that the run-off per square mile from them is greater than for the average of the total drainage area of the San Gabriel River, and that a larger percentage of it comes in extreme floods. However, it would be possible to handle economically a larger part of the floods from the smaller streams. Under the assumptions given in the two cases worked out, a diversion capacity of 2.25 sec-ft. per sq. mile of drainage area will assure sufficient supply for a reservoir storage of 200 acre-ft. for each square mile, irrigating 40 acres. At 2 cents per inch per hour this would warrant an expenditure of $13 250 per sq. mile of drainage area. In the same way, there would be available for increasing the underground storage a supply sufficient to irrigate 45 acres.

In many places where irrigation is developed extensively, the conditions of run-off are similar to those in the San Gabriel Mountains. Except in a few places, where large natural storage basins have been found on the streams, there is a large waste flood flow. As the value of the land and crops increases, the possibility of obtaining more storage increases. Much remains to be done to secure the maximum possible use of the water supply of almost all irrigated districts.

DISCUSSION

CHARLES H. LEE, ASSOC. M. AM. SOC. C. E. (by letter).—The author has presented in a very detailed and interesting manner the water supply situation on the San Gabriel River, and the writer agrees heartily with his conclusions regarding the possibility of increased use of the flow of Southern California streams by storage. The subject is one that offers an interesting field for engineering endeavor, and the author has laid the foundation for a broad discussion which should be of great value, not only to the Engineering Profession, but also to the inhabitants of Southern California. Mr. Lee.

The writer has been familiar with the water resources of Southern California for a number of years, and during the past year has had occasion to make detailed studies of three important streams, in connection with proposed increased use of the surplus flood. He has been impressed with one phase of storage in this region, which, although mentioned by Mr. Strong, was not particularly emphasized. This is the necessity for over-year storage in addition to monthly regulation, in order to accomplish the greatest beneficial use of stream flow. Mr. Strong has approached the subject with the primary idea of making increased use of the stream flow by regulation of monthly inequalities. The writer believes that permanent increased use of the surplus flow of Southern California streams requires the regulation of annual run-off as well as monthly flow.

TABLE 3.—TOTAL ANNUAL DISCHARGE OF SAN GABRIEL RIVER.

(Compiled from U. S. Geological Survey Records.)

Season, September 1st to August 31st.	Mean, in second-feet.	Acre-feet.	Percentage of mean.
1895-96	38.9	28 200	25
1896-97	126.4	90 500	79
1897-98	32.9	23 700	21
1898-99	13.8	9 900	9
1899-00	16.7	12 100	11
1900-01	137.0	95 400	82
1901-02	33.9	24 500	21
1902-03	145.3	104 800	92
1903-04	40.7	29 600	26
1904-05	220.3	158 700	139
1905-06	320.6	235 100	205
1906 07	483.9	349 200	305
1907-08	109.3	79 300	69
1908-09	254.3	179 500	157
1909-10	192.3	139 900	122
1910-11	376.0	271 600	237
Mean.........	158.9	114 500	100

The necessity for over-year storage is well shown by Table 3, which gives the total discharge of San Gabriel River for the 12-month season,

Mr. Lee. September 1st to August 31st, from the seasons 1895-96 to 1910-11. inclusive, as observed by the United States Geological Survey. It will be noted that during these 16 years the annual run-off has varied from a minimum of 9% of the mean to a maximum of 305%, or from one-tenth to three times the mean. Considering groups of three consecutive years, the extremes have been from 14 to 216%, or from one-seventh to more than twice the mean. The run-off during the 7-year period, 1897-98 to 1903-04, was only 37% of the mean, though that of the following 7 years averaged 176 per cent. Although the 3-year period of drought, 1897-98 to 1899-1900, is of infrequent occurrence, and might be tided over by temporary expedients, yet the benefit to be derived from annual storage is great, and the permanent increased use of stream flow cannot exist without it.

The determination of dependable supply with over-year storage regulation is easily made by constructing a mass-curve of daily discharge in acre-feet for the period of record. The ordinate to this curve at any date represents the aggregate run-off from the initial date of record to the date being considered, and the slope of the curve represents the rate of flow. The mass-curve can be constructed either for the full flow of the stream or for that portion which a transmission system of given capacity will divert. From such a diagram the safe yield of a given reservoir with the known water supply can be determined with precision, or, if desired, the storage capacity necessary for a given draft on the known supply. The writer has found such a curve to be the simplest and most comprehensive form in which to assemble the run-off data of a stream for solving the problems discussed by Mr. Strong, and suggests it as a more satisfactory method of ascertaining the supply available for storage than that used by the author.

The general absence of good reservoir sites of adequate capacity in the mountains eliminates the possibility of surface storage on most Southern California streams. Furthermore, surface storage is not desirable on account of the great evaporation loss. A reliable 4-year record of evaporation from a pan floating on the surface of Sweetwater Reservoir near San Diego indicates an annual depth of 58.7 in., amounting to about 40% of the annual storage. The conditions throughout the valley portions of Southern California, away from the Coast, are similar to those at Sweetwater Reservoir. The losses resulting from extended over-year storage in surface reservoirs in the valleys of Southern California, therefore, would tend to offset the benefit.

Underground storage possibilities in the gravel-filled basins which underlie these valleys have all the conditions necessary for efficient annual regulation. The storage capacity is almost unlimited, the evaporation losses outside the region of outlet are small, and the movement of water through the gravels to the region of outlet is so slow

Mr. Lee.

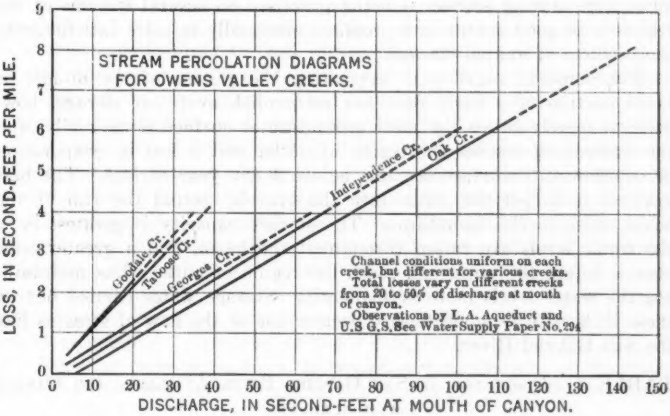


Fig. 20.

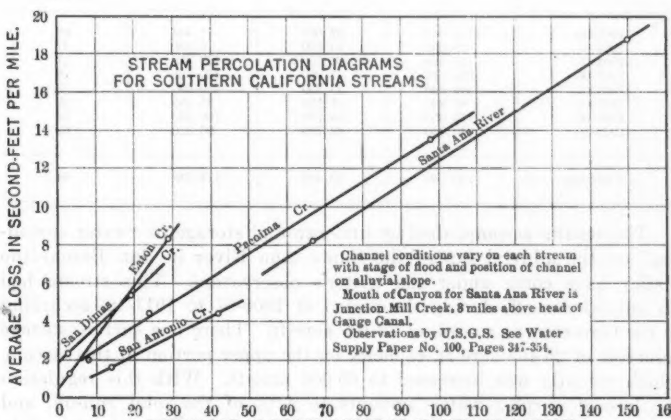


Fig. 21.

Mr. Lee.

that annual as well as monthly irregularities of supply are smoothed out. This type of storage is being practised on several streams of the region with good results and promises eventually to solve satisfactorily the problem of annual storage.

The common practice of saturating the gravels of the middle or lower portion of a basin does not accomplish over-year storage, however; it merely raises the local ground-water surface temporarily, and the water soon reaches the region of outlet and is lost by evaporation or overflow into surface streams before a dry year arrives. The best practice is to get the water into the gravels around the rim of the basin, close to the mountains. The storage capacity is greatest here, the water levels are raised throughout the basin, and a greater time elapses before the water can reach the region of outlet, thus maintaining the water levels more permanently. Storage water carried out on these lines will do much toward greater use of the flow of streams like the San Gabriel River.

TABLE 4.—Discharge of San Gabriel River Available for Canals and Surplus.

(Prepared from Author's Data.)

Season.	Total discharge, in acre-feet.	Discharge available for canals, in acre-feet.	Surplus.	
			Acre-feet.	Percentage of total.
1902–03	104 944	33 000	71 944	68
1903–04	29 650	24 500	5 150	17
1904–05	159 128	36 750	122 378	77
1905–06	234 776	44 050	190 726	78
1906–07	349 540	49 950	299 590	86
1907-08	83 200	47 550	35 650	43
1908–09	179 850	43 720	136 130	76
1909-10	140 040	43 730	96 310	69
Average......	160 140	40 400	119 740	64

The results accomplished by underground storage or "water spreading" on the alluvial fan of the Santa Ana River in San Bernardino Valley have come under the writer's observation. This stream had an annual run-off, during the period of 1900-01 to 1911-12, according to the Government record, of 72 100 acre-ft. There is a surface storage reservoir of 26 463 acre-ft. capacity on the upper portion of this stream, which recently was increased to 65 000 acre-ft. With this regulation the loss of surplus water amounts to 42% of the total run-off, and for the San Gabriel it is 64%, as indicated by Mr. Strong's data (Table 4). The conservation of this surplus by water spreading has been attempted for a number of years on a small scale, but during the last four seasons it has been placed on a permanent basis. From 10 000

to 15 000 acre-ft. annually are now being diverted and stored in the gravels of the San Bernardino Valley, of which about three-fourths are derived from Santa Ana River, thus reducing the lost surplus flow of that stream from 42 to 30% of the total run-off. The annual cost of this work is about 30 cents per acre-ft. stored, including operation and interest on investment. The quantity of water stored could be increased considerably with very little additional expense per acre-foot. Mr. Lee.

In connection with the subject of natural underground storage, the writer has had occasion to make careful and extended measurements of percolation from gravel stream channels in Owens Valley, and has found that, for stable channel conditions, a straight-line relation holds between loss in second-feet per mile of channel and total discharge (Fig. 20). Although, as the author states, the measurements of absorption on the San Gabriel River by the United States Geological Survey are not satisfactory, yet on other Southern California streams the data are more complete. Analysis of these data indicates that the straight line holds for percolation from the gravel stream channels of Southern California, for moderate discharges, where channel conditions are fairly stable (Fig. 21). The conditions on the Santa Ana River correspond closely to those on the San Gabriel, and give an idea of the extent to which absorption occurs through natural processes.

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Paper No. 1285

THE PREWITT RESERVOIR PROPOSITION.*

By J. C. Ulrich, M. Am. Soc. C. E.

The Prewitt Reservoir Proposition was conceived and developed by the Great Western Sugar Company for the purpose of increasing the effective water supply of the lower South Platte Valley, in the vicinity of Sterling. This Company, strictly speaking, is not engaged in the business of promoting irrigation propositions, but in the manufacture of sugar from beets. Though the promotion of irrigation propositions is not, in itself, one of its functions, it has a very definite indirect interest in the effectiveness of the water supply of the regions in which it operates.

Farmers will not raise beets, except at a profit, and beet culture cannot be conducted at a profit to the producer, except in regions in which a certain supply of water is available for use well into September.

This Company owns and operates three important sugar factories in the valley of the South Platte River: At Sterling, at Brush, and at Fort Morgan. The combined daily capacity of these plants, during the sugar-making campaign, is nearly 3 000 tons. In order that their operation may be profitable, it is necessary that they be supplied with a sufficient tonnage of beets to keep them running at least 100 days during each season. This requires about 300 000 tons of beets.

* A paper read before the Colorado Association of Members of the American Society of Civil Engineers, on January 11th, 1913, and presented before the American Society of Civil Engineers at the meeting of September 17th, 1913.

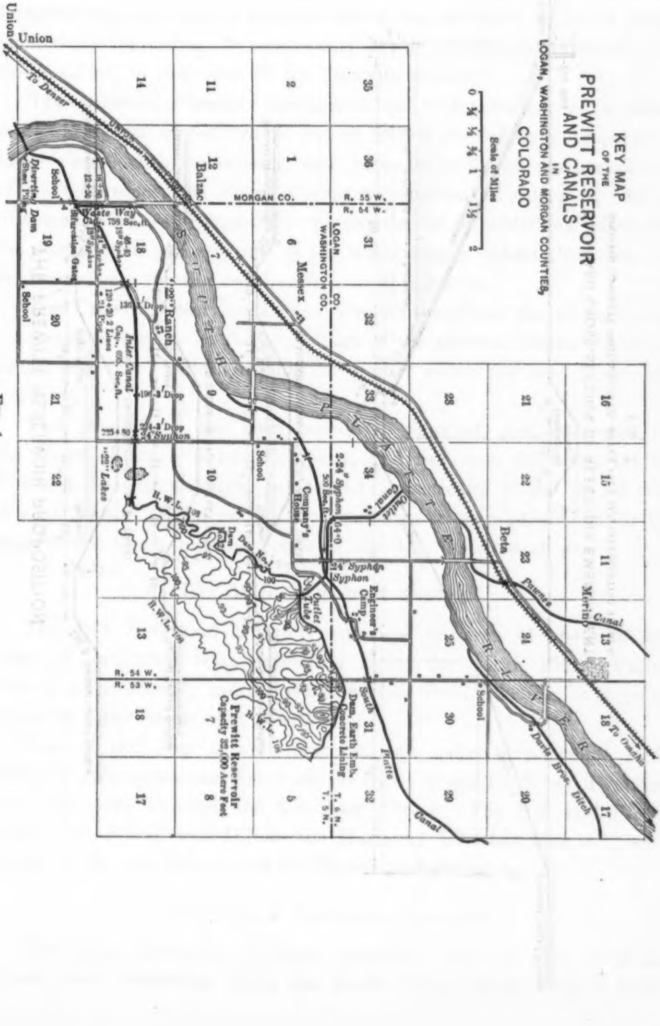
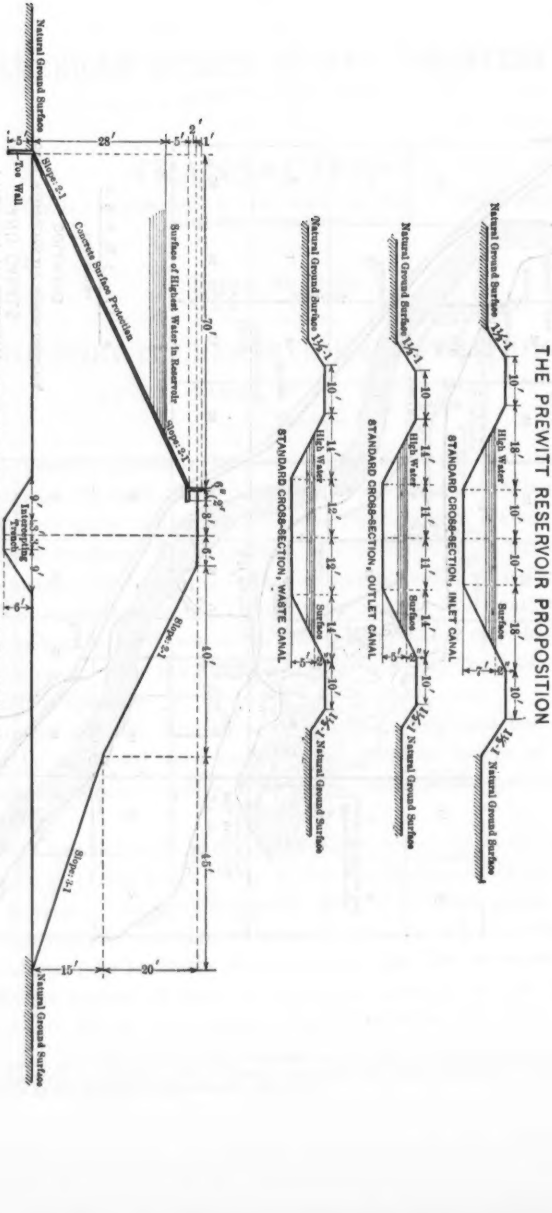

KEY MAP
OF THE
PREWITT RESERVOIR
AND CANALS
IN
LOGAN, WASHINGTON AND MORGAN COUNTIES,
COLORADO
Scale of Miles
0 ¼ ½ ¾ 1 1½ 2
MORGAN CO.
LOGAN CO.
WASHINGTON CO.
R. 55 W.
R. 54 W.
R. 53 W.
T. 6 N.
T. 5 N.
Union
To Denver
To Omaha
SOUTH PLATTE RIVER
Balzac
Messex
Beta
Merino
"22" Ranch
"22" Lakes
Waste Way
Diversion Dam
Bifurcation Gate
Inlet Canal
Outlet Canal
Outlet Tube
Pawnee Canal
South Platte Canal
Davis Bros. Ditch
Company's House
Engineer's Camp
School
Dam No. 1
H. W. L. 108
South Dam, Earth Emb. Concrete Lining
Prewitt Reservoir Capacity 33,000 Acre Feet
24" Syphon
2-24" Syphon

FIG. 1.



STANDARD CROSS-SECTION OF RESERVOIR EMBANKMENT

SHOWING CONCRETE PROTECTION ON WATER FACE AND INTERCEPTING TRENCH

FIG. 2.

After these sugar plants were constructed, it was found that, in some years, the water supply was not sufficient, during the later part of the growing season, to effect a full yield of beets; the result was that the supply of the latter was reduced, and the operating time of these factories was curtailed to a period which was too short to give a profit from their operation. In one season it was even found necessary, on this account, to close one of the factories entirely.

The enforced idleness, throughout an entire season, of a plant representing an investment of nearly $1 000 000 is not a proposition to be viewed with equanimity, and, in order to lessen the probability of its recurrence, the Sugar Company conceived the idea of adding to the effective water supply by the creation of an additional plant for the storage and conservation of the water supply existing but not yet made available for the requirements of this region.

The Prewitt Reservoir Proposition contemplates the storage, and conservation for use in the latter part of the growing season, of about 32 000 acre-ft. of water, in addition to that which has been heretofore available.

This proposition has been conceived, financed, and developed by the Great Western Sugar Company, and has been delivered to the Iliff and the Logan Irrigation Districts, practically at the actual cost of construction of the works; the Company, in reimbursement for its expenditure, accepting the bonds of the two districts at par.

The Iliff Irrigation District.

The Iliff Irrigation District comprises an area of about 13 600 acres of cultivated land, extending along the South Platte Valley from a point about 7 miles down the river from Sterling, to a point about 32 miles below that town.

Most of this area is well improved and under cultivation. It is irrigated at present, and has been for many years, by direct diversion from the river, through the following ditches: The Iliff and Platte Valley, the Powell and Dillon, the Harmony (Ditches Nos. 1, 2, and 3), the J. B., the Bravo, and the Powell and Bland.

The Logan Irrigation District.

The Logan Irrigation District comprises, also, an area of about 13 600 acres, extending along the South Platte River from a point

about 14 miles up the river from Sterling, to a point about 6 miles down stream from that town.

Like the Iliff District, this area is well improved, and has been under cultivation for many years. Its water supply is dependent on direct diversion from the river, through the following ditches: The Pawnee, the Springdale, the South Platte Extension, and the Davis Brothers.

The territory in both these districts is well improved, and has enjoyed a reasonable degree of prosperity, without the advantage of an impounded water supply. It is hoped that this prosperity will be increased materially by the more certain water supply which is expected under the operation of the proposition here under consideration.

The Prewitt Reservoir Proposition.

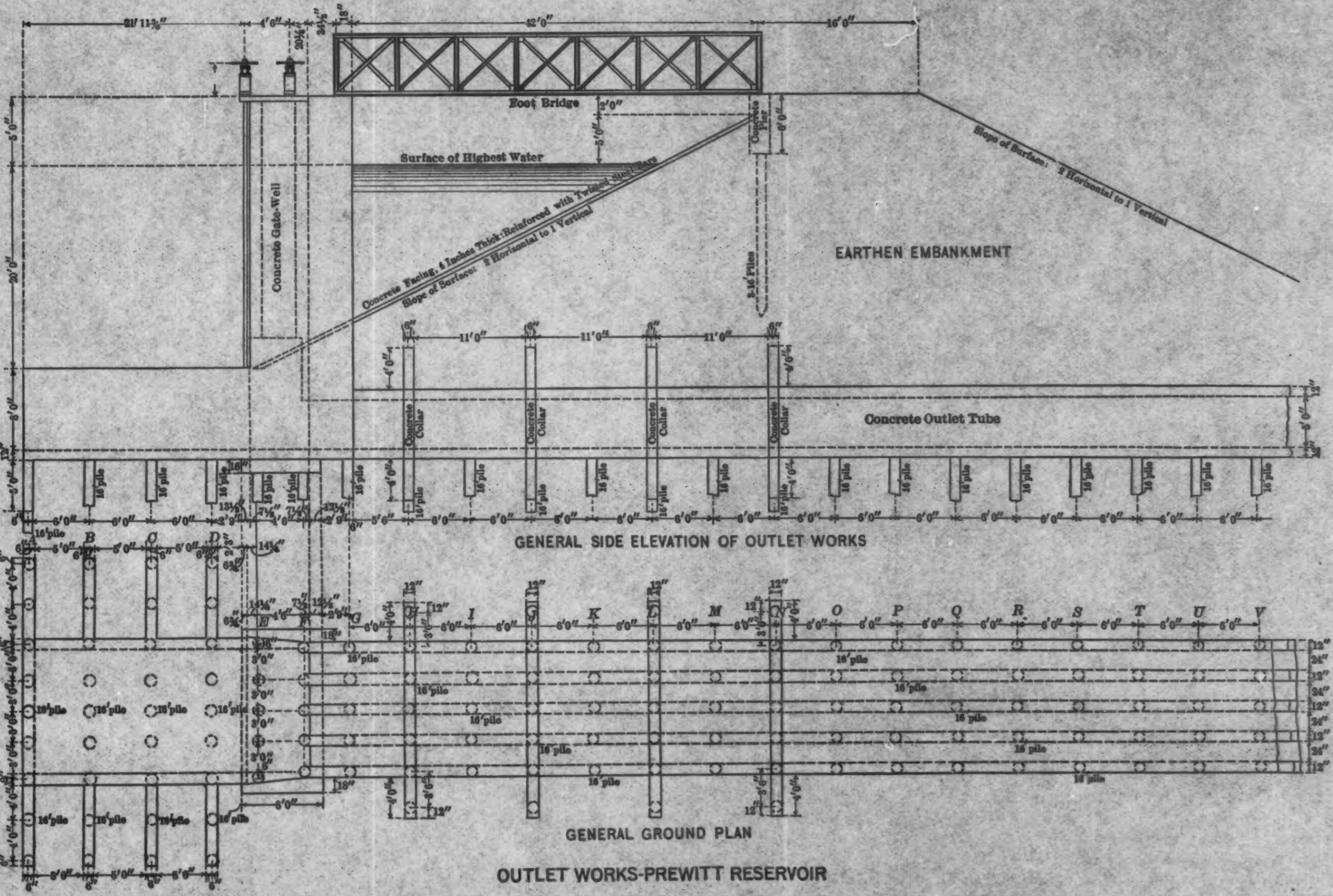
The Prewitt Reservoir Basin consists essentially of a shallow sandy depression on the plains bordering the South Platte River. It is about 2½ miles from the South Platte River, about the same distance from the Town of Merino, on the Julesburg Division of the Union Pacific Railroad, and 13 miles southwest from Sterling.

Its improvement has consisted of the construction of an earthen embankment, about 3½ miles long, occupying about one-third of its contour; the construction of an inlet canal, 5 miles long, from the river, and an outlet canal, 2 miles long, designed for returning the impounded waters to the river for re-diversion through the ditches previously named, and application to the lands of the districts.

The Reservoir Embankment.—The reservoir embankment has a maximum height of 36 ft. This height is reached at only one place, and extends for a distance of less than 100 ft. For the greater portion of its length, the height does not exceed 25 ft., the average being about 20 ft.

It is designed with uniform slopes of 2 horizontal to 1 vertical, on the water side. Where the height does not exceed 20 ft., the outside slopes are also uniformly 2 horizontal to 1 vertical. Where the height exceeds 20 ft., the outside slopes, from the top of the embankment to an elevation 20 ft. below the top, are, likewise, 2 horizontal to 1 vertical, and, from this elevation to the base of the fill, they are 3 horizontal to 1 vertical. The top width, throughout, is 16 ft., and the crest is 7 ft. above the level of the highest proposed water in the reservoir.

PLATE III.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVII, No. 1285.
ULRICH ON
PREWITT RESERVOIR PROPOSITION.



OUTLET WORKS-PREWITT RESERVOIR

FIG. 3.—PLACING CONCRETE PROTECTION SLABS ON FACE OF RESERVOIR EMBANKMENT.

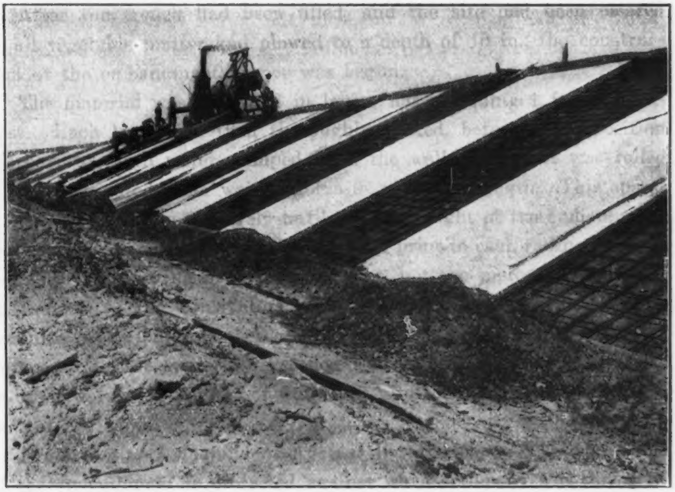


FIG. 4.—COMPLETED SLABS, AND REINFORCEMENT READY FOR CONCRETE FOR ALTERNATE SLABS.

The material on which the embankment is founded, and of which it is constructed, consists essentially of very fine sand mixed with a small percentage of soil.

Prior to the construction of the embankment, a longitudinal trench was excavated along the entire site, its center line being coincident with that of the embankment. This trench was made 6 ft. deep and 6 ft. wide on the bottom, with side slopes of $1\frac{1}{2}$ horizontal to 1 vertical.

Before depositing any earth for the embankment proper, this trench was partly filled with water, in which selected material was deposited in 2-ft. layers. This operation was repeated three times in the filling of the trench. The water for this purpose was pumped from a series of sixteen wells, put down just outside of the lower toe of the embankment, at intervals of about 1 000 ft. Sufficient water was thus furnished and used, to effect, not merely the moistening, but the actual puddling, of the material deposited in the trench.

The purpose of this puddled trench was to break the continuity of any seam which there might be between the soil of the site and the material of the superimposed embankment. It was also designed to cut off and intercept the channels of any dog or gopher holes which might be in the material underlying the embankment.

After the trench had been filled, and the site had been cleared of all vegetable matter and plowed to a depth of 10 in., the construction of the embankment proper was begun.

The material was deposited in layers not exceeding 1 ft. in thickness. Each layer was then thoroughly wetted, before the deposition of the next, with water pumped from the wells. Then it was rolled with a corrugated roller weighing 125 lb. per in. of length. This operation was repeated successively until the full height of the embankment was reached. The wetting of this material prior to each rolling resulted in the actual wetting of the whole layer, not the mere moistening of the surface. The contractors claim to have kept records of their pumping operations, and these disclose the fact that the volume of water pumped into the material exceeded that of the embankment itself; in other words, the volume of water put into the embankment exceeded that of the earth.

Protection of Embankment Against Wave Action.—The water side of the embankment is protected against wave action by a covering of

concrete, 4 in. thick, extending from its foot to within 2 ft. of its top, where it joins a vertical parapet wall of concrete. The latter is 3 ft. high, and extends 1 ft. above the crest of the embankment, and 8 ft. above the elevation of the highest proposed water in the reservoir. This parapet is 6 in. thick, and is L-shaped, with a vertical leg of 3 ft. and a horizontal leg of $2\frac{1}{2}$ ft. Both legs are reinforced in each direction with $\frac{3}{8}$-in. square, twisted bars at intervals of 12 in.

At the foot of the surface protection, and connected therewith by reinforcing rods of steel, there is a vertical "toe-wall," extending 5 ft. into the ground below the edge of the latter. This toe-wall is 6 in. thick, and is reinforced horizontally and vertically with steel bars at intervals of 12 in.

The concrete sheathing which covers the surface of the embankment is in individual slabs, each 10 ft. wide, and continuous in length from the parapet wall at the top to the toe wall at its foot.

These slabs were laid in two distinct sets, the first set being placed with intervening spaces of the same widths as the slabs themselves. After the first ones had become thoroughly set, the alternate slabs were placed. Side forms were placed for the first set of slabs, but their edges constituted the forms for the alternate set.

Under each line forming the junction of adjacent slabs, and designed for the purpose of breaking the joints between the latter, there are concrete stringers, extending from the top to the bottom of the inclined surface to be protected. These stringers are 6 in. thick and 12 in. wide, and each is reinforced with three bars of $\frac{3}{8}$-in. square, twisted steel, placed midway between the upper and lower surfaces of the stringer. The edges of adjacent protection slabs are coincident with the center line of these stringers, each slab lapping over the stringer 6 in. It was expected that this arrangement would break the joints between the edges of the slabs, and prevent the removal of sand and earth through seams which might be caused by the contraction of the slabs.

The slabs are reinforced in both directions with $\frac{3}{8}$-in. square, twisted, steel rods at intervals of 12 in. Great care was taken to ensure that these rods would be midway between the two surfaces of the concrete. The longitudinal and cross-rods were secured to one another with wire fastenings at each alternate intersection.

The concrete in all this revetment was composed of 1 part of Ideal

FIG. 5.—TOE-WALL IN THE FORMS.

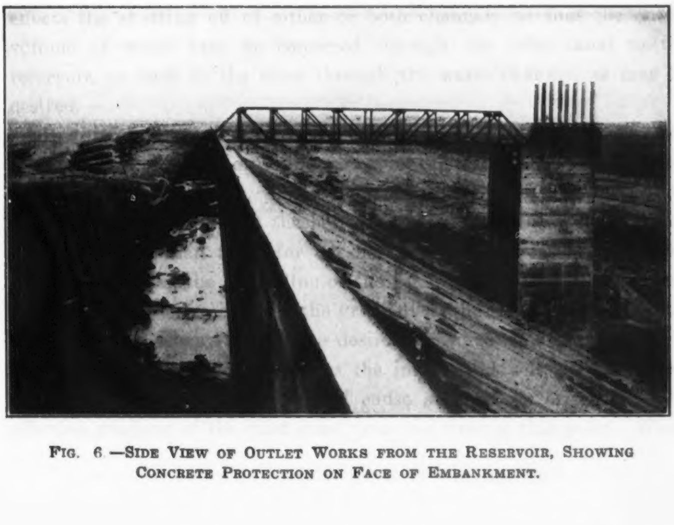


FIG. 6.—SIDE VIEW OF OUTLET WORKS FROM THE RESERVOIR, SHOWING CONCRETE PROTECTION ON FACE OF EMBANKMENT.

Portland cement and 4 parts of clean, sharp, river sand. Gravel was not used because none was obtainable within permissible transportation limits.

The Inlet Canal.—The inlet canal, conveying the water from the river to the reservoir, is 5 miles long, and is on a gradient of 0.025 per 100, equivalent to about 16 in. per mile, the computed velocity being 2.9 ft. per sec., with the canal running full.

The canal has a uniform bottom width of 20 ft., and side slopes of 2 horizontal to 1 vertical. It is built through a comparatively level country, and has an embankment on each side. The top width of these embankments is 10 ft., and the crest elevation is 9 ft. above the bottom of the canal and 2 ft. above the elevation of highest water.

The canal is located mostly on tangents, there being only six curves, of which the sharpest is 6 degrees. Its computed discharge, for a depth of 7 ft., is about 695 cu. ft. per sec.

About 1 mile below the point where the canal is diverted from the river, a waste channel extends back to the river, a distance of 1 mile. This has been designed to assist in removing sand from the canal. It has a bottom width of 24 ft., side slopes of 2 to 1, and is designed to carry water to a maximum depth of 5 ft. It has an embankment on each side with top widths of 10 ft. and crest elevations 7 ft. above the bottom of the channel.

At the origin of this waste canal there is a double structure which effects the shutting off of either or both channels, so that the entire volume of water may be conveyed through the inlet canal to the reservoir, or back to the river through the waste channel, as may be desired.

In this structure there are two sets of regulating gates, one across each channel. There are nine gates in each set, each gate serving to close an aperture 4 ft. wide. The tops of the gates in each set are at the same elevation, but the bottoms of those for the waste channel are 2 ft. lower than those for the inlet canal. The latter are at the same elevation as the grade line of the inlet canal. This arrangement was made in order to increase the gradient in the first mile of the inlet canal at times when it might be desired to sluice sand out of it. At such times, the check-gates across the inlet canal will be closed, and the waste-gates opened, which will cause an increase of 2 ft. in the effective gradient of the inlet canal from the river to this point. When

the sluicing operations are concluded, the waste-gates will be closed and the check-gates opened, thus restoring the normal gradient in the inlet canal.

Structures on the Inlet Canal.—The head-gate at the origin of the inlet canal is of concrete, reinforced with ½-in. square, twisted, steel bars, and is founded on round piles, driven 12 ft. into the ground. It has nine 4 by 9-ft. apertures, closed by steel gates operated by rising screw stems, 2½ in. in diameter at the root of the thread. The gates are raised and lowered by hand-wheels, operating on steel balls running in bronze grooves.

The sides, partitions, and floors are 12 in. thick, and are reinforced with ½-in. square, twisted, steel bars, in horizontal and vertical pairs, at intervals of 12 in., making 4 lin. ft. of reinforcing rod to each cubic foot of concrete.

The double structure at the point where the waste canal originates is composed of two parts, each in all essential respects similar to the head-gate just described, except that the heights of the gates differ, and the waste structure has a drop of 3 ft. beginning a short distance behind the gates, thus combining the functions of a waste-gate and a drop.

There are four concrete drops on the inlet canal, designed for the purpose of accommodating it to the fall of the country in excess of that required for the gradient of the canal. Three of these lower the grade of the canal by 3 ft. each; the fourth is at the terminus of the inlet canal, where it enters the reservoir basin, and consists of two successive drops of 5 ft.

In all cases these drops consist of reverse circular curves in vertical planes, and the sides of the structures conform to the section of the canal, with slopes of 2 horizontal to 1 vertical. Each has five cut-off walls extending below the floor and sides to depths of 5 ft. or more. These walls rest on round piles, driven 12 ft. into the ground. The sides, floors, and cut-off walls are in all cases 12 in. thick, and are reinforced in the manner described for the head-gate structure, involving 4 lin. ft. of steel rod for each cubic foot of concrete.

There are nine bridges crossing the canals of this system. These are all steel spans on concrete abutments, no obstructions being permitted in the channels, aside from the partitions in the head-gate and bifurcation structures, which could not be avoided.

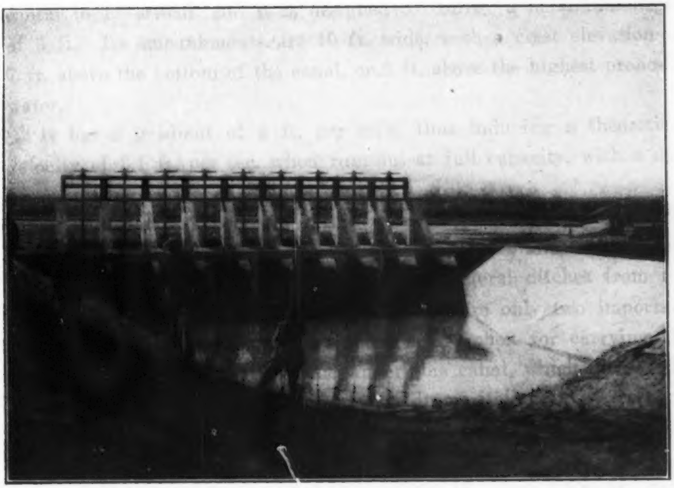


FIG. 7.—REAR VIEW OF HEAD-GATE STRUCTURE, HEAD OF INLET CANAL.

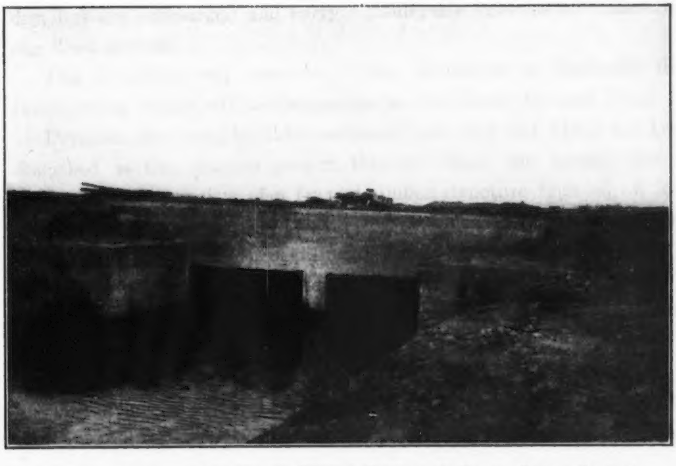
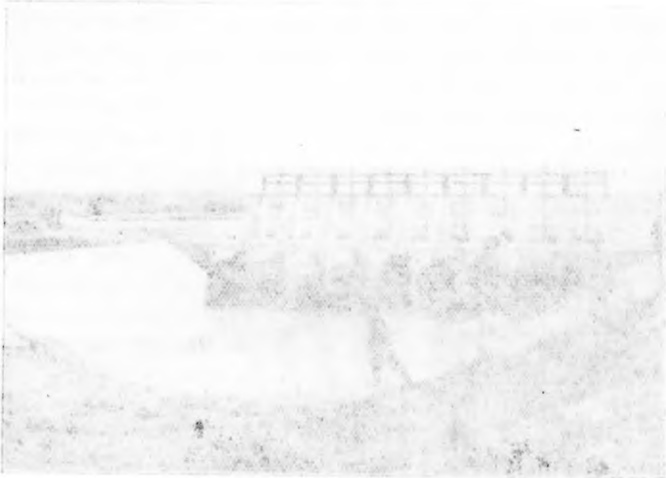


FIG. 8.—CONCRETE CULVERT OR SIPHON CONVEYING CANAL UNDER OUTLET CANAL FROM RESERVOIR.



The Outlet Canal.—The outlet canal from the reservoir to the river is 2 miles long. Its bottom width is 22 ft., its side slopes are 2 horizontal to 1 vertical, and it is designed to convey a maximum depth of 5 ft. Its embankments are 10 ft. wide, with a crest elevation of 7 ft. above the bottom of the canal, or 2 ft. above the highest proposed water.

It has a gradient of 2 ft. per mile, thus inducing a theoretical velocity of 3.1 ft. per sec. when running at full capacity, with a discharge of about 525 cu. ft. per sec.

In addition to a concrete rating weir, a distribution gate for delivering water to the South Platte Extension Ditch, and two concrete siphons for conveying the water of certain lateral ditches from the South Platte Ditch under its channel, there are only two important structures on this line. One of these is a siphon for carrying the waters of the South Platte Ditch under this canal, which crosses the line of the latter; the other is a 5-ft. drop, similar in all respects to those already described.

The River Diversion Weir.—The river channel, at the point of diversion of the inlet canal, has a width of about 1 500 ft., from bank to bank. Only about 400 ft. of this distance, however, carries water during ordinary stages of the river. The remainder consists of low-lying lands traversed by minor channels, which are ordinarily dry, but are submerged and carry considerable volumes of water during flood periods.

The diversion weir consists of two structures of distinctly different types, which will be designated as Divisions One and Two.

Division One occupies that portion of the river bed which has been described as the channel proper, through which the normal flow is discharged. It consists of a framed timber structure founded on four rows of round piles driven 12 ft. into the sands of the river bed, and two rows of Wakefield sheet-piling designed as cut-off diaphragms for preventing the passage of water under the structure. Each piece of the Wakefield sheet-piling, consists of three pieces of 2 by 12-in. plank, 12 ft. long, fastened with 60-penny wire spikes, in the form of a tongued and grooved member.

This structure has on its top a movable device which can be elevated or depressed, as may be desired. In periods of very high water, when

the adjoining lands would be in danger of overflow, it is intended that this movable superstructure will be depressed, giving a larger waterway for the passage of flood-waters. At other times, when the flow is normal, it will be elevated, raising the surface to the elevation of highest water in the canal.

The crest of the fixed structure is about 2½ ft. above the natural surface of the river channel and 5 ft. above the floor of the head-gate of the inlet canal, the latter being 2½ ft. below the natural bed of the stream. The movable device permits of the elevation of the effective crest by 2 ft.

This division of the diversion weir is provided with two sluice-ways, regulated with flash-boards, which when opened permit the water in front of the weir to be lowered to an elevation 2 ft. below the crest of the fixed structure. One of these is at the end of the weir next to the head-gate; the other is near the middle of the portion designated as Division One. Each is 30 ft. wide and is divided into six 5-ft. openings, separated by steel girder diaphragms designed for supporting flash-boards. The length of this division of the structure is 435 ft.

Division Two of this structure occupies, and closes permanently, that part of the channel which in ordinary stages of the river is practically without water. It consists essentially of an earthen fill, 20 ft. wide on top, with 2 to 1 side-slopes, and protected on the up-stream side with concrete in the same manner as the reservoir embankment, except that, in this case, there is no vertical parapet wall at the top of the protection.

For the purpose of preventing the passage of water through the sands of the river bed under this embankment, a line of Wakefield sheet-piling 14 ft. long was driven along its center line from one end to the other. This sheeting was driven to depths varying from 10 to 12 ft., and its upper ends project above the surface, to heights varying from 2 to 4 ft., and thus extend into the embankment.

The total length of this structure is 980 ft. Its connection with Division One is effected by a concrete wall which rests on piles and is buttressed on the side in contact with the earthen structure. The elevation of the top of this structure is 7 ft. above the fixed crest of Division One, or 5 ft. above the top of the movable superstructure thereof when the latter is erected.

PLATE IV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVII, No. 1285.
ULRICH ON
PREWITT RESERVOIR PROPOSITION.

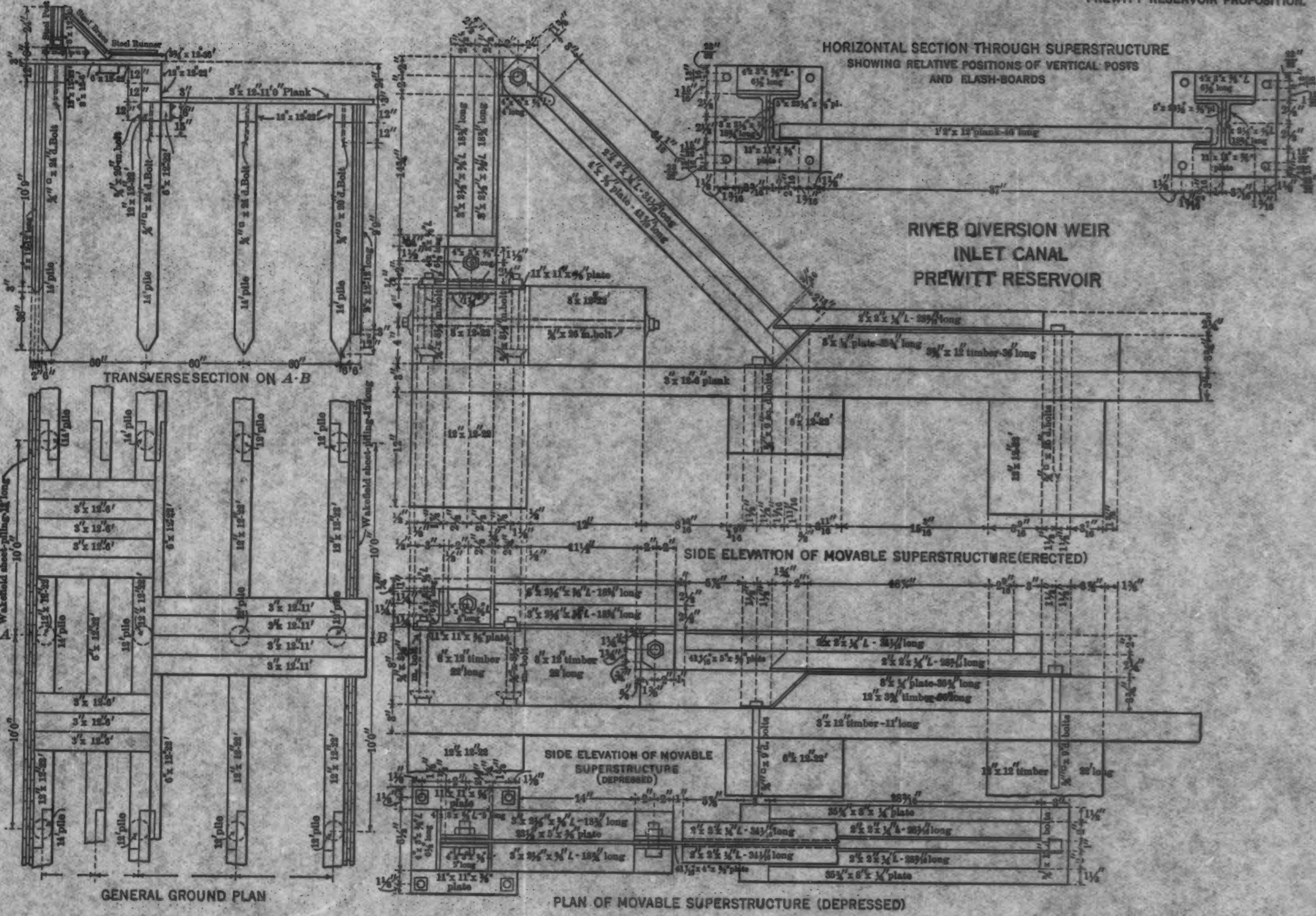


FIG. 9.—THREE-FOOT DROP IN INLET CANAL TO RESERVOIR.

FIG. 10.—DIVERSION WEIR IN RIVER AT HEAD OF INLET CANAL.

General Comments.

It is not claimed for this proposition that it embodies any remarkable or unique features of unusual interest or superiority. The basin itself is by no means typical, its area being out of proportion to its depth. It occupies a site which is very sandy, and it is expected that much water will be lost by percolation and seepage during the early years of its operation.

Its unusual extent of embankment (3½ miles in length), constructed almost entirely of fine sand, and founded on the same material, carries with it a definite liability which cannot be entirely ignored, even though the danger does not appear to be imminent.

In its extremely short inlet of 5 miles, and its outlet to the river of 2 miles, both constructed through a comparatively level country involving no physical problems of difficulty, it has, however, a natural advantage which is not found, to a like degree, in any similar proposition in the South Platte Valley.

The feature of merit which is claimed for this proposition, as distinguishing it from most others heretofore developed by private enterprise in this region, is the substantial and permanent character of its structures.

Aside from the diversion weir across the river at the origin of the inlet canal, there is not a structure in the work which is not executed substantially of reinforced concrete or steel.

All these structures have been built under specifications which were more than ordinarily exacting, and the work has been executed in more definite conformity to the specifications than has been customary in most developments of this character.

With reference to the probable permanence and effectiveness of the works, there appear to be no reasonable grounds for apprehension, aside from the uncertainty that must be faced as to the possibility of protecting effectually an earthen embankment of friable material against the destructive effects of wave action during the high winds which frequently prevail in the plains regions during the spring.

In the writer's opinion, absolute safety, under these circumstances, cannot be predicated of an earthen embankment constructed of material readily responsive to erosion, the protection of which depends merely on a sheathing of concrete covering a material which, in itself, has no power of resistance against the erosive action of moving water.

Though the concrete protection here under consideration has been executed more effectively than can be claimed for most works of this character with which the writer is acquainted, and though its design embodies every precaution that he was able to devise, within the limits of expense that could be borne by the enterprise, he cannot escape the conviction that definite uncertainty exists as to the entire efficiency of this and other works of like character, created with the same end in view.

The development of this proposition involved the placing of 16 700 cu. yd. of concrete, in the composition of which 121 000 sacks of cement, and 660 tons of reinforcing steel were used. In addition to the reinforcing steel, about 90 tons of structural steel were used in building the structures, exclusive of the nine steel bridges crossing the different canals. The total expenditure was somewhat less than $700 000.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

Paper No. 1286

ROAD CONSTRUCTION AND MAINTENANCE.

AN INFORMAL DISCUSSION PRESENTED AT THE MEETINGS OF JANUARY 17TH AND 18TH, 1913.

PAGE

(1) CEMENT-CONCRETE PAVEMENTS.

By Messrs. J. A. Johnston, A. H. Blanchard, Sanford E. Thompson, E. H. Thomes, Philip P. Sharples, L. P. Sibley, Samuel Whinery, Harold Parker, William M. Kinney, and James Owen.

Mr. Johnston.

J. A. Johnston, M. Am. Soc. C. E.—Great care is requisite in order to obtain good results in concrete pavements. In discussing the value of such pavements, there is, of course, some question as to the kind and amount of traffic they will stand. In Massachusetts, in 1906, a concrete surface was laid on a section of State highway in the Town of Spencer, and stood the traffic remarkably well. The principal objection was that it was rather hard for the feet of the country-bred horses, which, if allowed to select the traveled way, would turn out on the earth shoulders. A bituminous surface remedied this condition, and at present the road is giving excellent service. The section mentioned was put down by the grouting method, known as the Hassam process. On investigation it was found that the grout had penetrated, not only 6 in. of broken stone, but an additional 2 in. of gravel sub-surface. Very little trouble was experienced from disintegration of the concrete surface; it was confined principally to the area adjacent to the expansion joints.

Since that time some roads have been built by mixing the concrete before placing. Care had to be taken to have the batch properly tempered, particularly in building on a grade. If too wet, the mortar had a tendency to creep and form waves on the surface, and, of course, a dry mixture was not readily flushed, hence it was difficult to get a good smooth surface. The concrete was placed in a single layer, and as near a mosaic surface as possible was desired, in order to permit the bituminous material to bond properly. To obtain that result various methods were tried. After placing the concrete, dry stone was spread over it, and tamped lightly. This worked well, but required considerable skill on the part of the men who were spreading the broken stone. The experiment of allowing the concrete to set for 24 hours, and then brushing it off with a steel broom, gave, perhaps, the most satisfactory results.

There is a tendency to use too lean a mixture for concrete road surfaces. Many engineers have advocated a 1:3:6 mixture, but that is not rich enough, for concrete exposed to the traction of wheels and blows from horses' hoofs must be very tough to resist abrasion. Added strength cannot be gained by reducing the percentage of stone aggregate, that is, assuming 2 parts of sand to be the correct quantity to be used with 1 part of cement. Such a mortar will fill the voids in 4 parts of stone, and no added strength is gained by reducing the stone

aggregate to 3 parts. Too small a proportion of stone means a large percentage of mortar, thus giving a smooth surface, which would be slippery and not desirable. Mr. Johnston.

The concrete pavement has the advantage of being not only a surface, but also a foundation. Though, of course, there may be conditions where some treatment of the sub-surface will be needed, with a uniform soil, in which the frost action is about the same in all parts, there is very little difficulty. If, however, the soil is varied, so that the frost will affect one portion of the road more than another, the concrete will crack. It is an unusual condition where such cracks are serious enough to disintergrate the concrete, and any ordinary cracking is not material. This is particularly true where concrete is coated with a bituminous material, as the cracks are then sealed over and give no trouble. With concrete, as with all other smooth paving, the surface is rather slippery, necessitating care in designing the maximum grades and the camber of the road.

A light bituminous top on a cement-concrete base solves many road problems. It is reported that there is much difficulty in making the bituminous material stick to the concrete. In Massachusetts success has been obtained by the following method: The concrete surface is swept as clean as possible, then sprinkled with water, and while still wet covered with Tarvia A, heated to 200° Fahr. and applied at a pressure of not less than 70 lb. per sq. in., and at the rate of ½ gal. per sq. yd. of surface. This is then covered with clean stone screenings (not exceeding ½ in. in diameter and from which the flour has been removed) spread at the rate of 0.015 tons per sq. yd. of surface. This is again watered, and while still wet a second ¼-gal. coating of tar is applied in the same manner as before, but covered with clean, gravelly sand, using 0.015 cu. yd. per sq. yd. of surface.

The speaker is aware that the application of tar on a wet surface is contrary to existing theories, but it has worked satisfactorily. This road, treated in August, 1910, has cost to date, $15 for total repairs on more than 4 000 lin. ft. of 15-ft. surface, and it has taken only 30 gal. of tar to replace such small places as have scaled off. The speaker believes the successful adhesion of the tar to the concrete is due to the wet surface of the latter and the high pressure used in spraying the tar on the road.

A. H. Blanchard, M. Am. Soc. C. E.—Although cement-concrete pavements have been used in the United States since 1893, various details of construction and maintenance are as yet in an experimental stage. Many miles of cement-concrete pavements have been examined by the speaker, and the only ones which have been observed to be free from cracks are those which have been carefully constructed by the mixing method, and built on a well-drained, thoroughly compacted foundation, with adequate transverse expansion-contraction joints. Such Mr. Blanchard.

Mr. Blanchard.

joints are necessary, if successful results are to be expected, and both longitudinal joints along the curbs and transverse joints should be provided. If there are no expansion-contraction joints, the tensile strength of the concrete will probably be exceeded when the concrete contracts, and the pavement therefore will crack; when it expands, it will tend to bulge, and if the expansion produces forces which are in excess of the compressive strength of the concrete, the latter will crush at the cracks. It is obvious that the edges of the joints should be protected from the abrasive action of traffic, and it is likewise apparent that the joints should be filled with a material which will provide for movement between the joints as the pavement expands and contracts.

The maintenance of cement-concrete pavements is difficult, as it is necessary to keep traffic off the freshly laid patches. It would seem advisable to use a bituminous concrete for patching, and the bituminous cement should be of such a character that the bituminous concrete will be stable after thorough compression by tamping or otherwise. Naturally, a cement-concrete pavement repaired in this manner will not be pleasing from an esthetic standpoint. However, this point is not one of weight, as in all probability a large percentage of cement-concrete pavements constructed in the future will be finished with a bituminous surface.

Mr. Thompson.

SANFORD E. THOMPSON, M. AM. SOC. C. E.—One of the problems to be considered in the construction of concrete pavements is that of expansion and contraction due to changes in temperature. Contraction caused by the lowering of the temperature produces cracks, not only in concrete which is not reinforced, but also in brickwork or stonework. In the latter, however, the cracks are not apt to be so noticeable, because they follow the line of the joints.

Although it is possible to reinforce concrete pavements with steel so that the size of the cracks is inappreciable, the introduction of steel in sufficient quantity for this purpose is somewhat questionable from the standpoint of cost, and the problem, therefore, resolves itself generally into the location of the cracks where they are wanted, so that they will have smooth joints instead of rough, irregular lines.

Longitudinal joints through the middle of the pavement should be avoided if possible. Sometimes, it is advisable to make a longitudinal joint on each side of the pavement next to the curb. The spacing of joints across the pavement depends on climatic conditions and the extremes of temperature that will occur in the given locality. A spacing of from 20 to 30 ft. is common practice in first-class construction. If the concrete is laid in cold weather, at a temperature not very much above freezing, its tendency to contract is less, and the cracks will be apt to occur at a greater distance apart. In some cases, this fact may be taken into account in locating the cross-joints.

Mr. Thompson.

It is possible to calculate quite closely the actual amount of contraction which will take place during a falling temperature, and to determine the approximate size of crack that is likely to open. The coefficient of expansion of concrete is approximately 0.0000055; that is, for every degree of fall in temperature, the concrete contracts in this ratio. Suppose, for example, a section of concrete is 50 ft. long, and that it is free to expand and contract, the actual contraction caused by a fall in temperature of 40° Fahr., theoretically, would be $0.0000055 \times 40 \times 50 = 0.011$ ft., which is about $\frac{1}{8}$ in. In several cases the speaker has had occasion to check this theoretical contraction in actual concrete structures, and has found that it agrees closely enough for practical purposes. In one case, for example, he measured the contraction in a long, reinforced concrete, freight shed in which joints had been left at intervals; another case was in a large reservoir bottom, which was free to expand on one side; a record of the tests in the Croton Dam has been published.*

The necessity for frequent joints, in the first place, is to keep them small in size, and, in the second place, to prevent irregular cracks from opening up between them. There are certain patents on the market for steel-bound joints, which prevent chipping at the edges. It is a question, however, whether it is not more economical to place the joints at fairly close intervals, except where the road traffic is very heavy.

The question of expansion requires very little consideration. Many concrete pavements are built with open joints filled with a bituminous mixture to permit expansion. In ordinary cases, where the pavement is on a uniform grade, this is not necessary, because any expansion will be taken up by the concrete itself, usually by producing compressive stresses.

Many failures of concrete pavements are due to the poor quality of the aggregates. Not only must the sand of the aggregate be clean in appearance, but it also should be subjected to actual test. Sand which passes even close inspection has been found in a great many cases to fail to set properly in mortar or concrete. A very small, almost insignificant, quantity of vegetable matter in the sand—a quantity which cannot be determined by the eye—may prevent the concrete from hardening. An especially good, fine aggregate is particularly necessary in the wearing surface of a pavement, because of the extremely hard usage to which it may be subjected. It is absolutely necessary, therefore, in any pavement used as a wearing surface, to test the sand. A mechanical analysis test is of value to determine the quantity of very fine material, which is especially detrimental to a wearing surface. In addition, however, there should always be a test of the strength of the mortar, made up into briquettes, or cubes, in the proportion of 1 part cement to 3 parts sand, and these specimens should

* *Transactions*, Am. Soc. C. E., Vol. LXI. p. 399.

Mr. Thompson. be compared with similar ones made with the same cement and standard Ottawa sand. For the fine aggregate for a wearing surface, the strength of the mortar containing the sand in question should equal that of the standard sand mortar.

Mr. Thomes. E. H. THOMES, M. AM. SOC. C. E.—The results obtained with the oil-cement, concrete section of the experimental pavements constructed on Hillside Avenue, Jamaica, N. Y., in August, 1911, by the Borough of Queens, in co-operation with the United States Office of Public Roads, may be of interest.

About May, 1912, a number of depressions from 1 to 2 ft. in diameter, and from ½ to 1½ in. deep, had developed in the surface. On account of the traffic, it was impracticable to make repairs with Portland cement, and, therefore, this section was covered with a bituminous seal coat of about ⅜ gal. per sq. yd., applied by Tarrant hand-pouring pots and covered with sand and screenings (0.01 cu. yd. per sq. yd.). The whole section was first thoroughly cleaned by hand-brooming—ordinary, factory brush brooms—and the larger depressions and expansion joints were painted with tar asphalt, composed of Tarvia A mixed with about 15% of oil asphalt. A layer of ¾-in. stone was placed in the deeper, and one of dustless screenings in the smaller, depressions. The whole surface was then covered with a seal coat. Tarvia A was used from Station 0 to Station 0 + 48; tar asphalt from Station 0 + 48 to Station 1 + 52 and from Station 1 + 67 to Station 1 + 73; and Texas asphalt 55 from Station 1 + 52 to Station 1 + 67. The asphalt was applied first in a thin paint coat of 1 part asphalt to 2 parts gasoline, and then covered with the asphalt cement. Sand covering was used from Station 0 + 24 to Station 0 + 80, screenings from Station 0 to Station 0 + 24 and from Station 0 + 80 to Station 1 + 28, and dustless screenings or stone chips from Station 1 + 28 to Station 1 + 73.

In January, 1913, the Tarvia A had scaled off in a few spots, the tar asphalt was about one-third worn off, and the asphalt about one-fifth gone. There is very little apparent difference in the results obtained with the sand, screenings, and stone chips. The sand appears to give the same result as the stone chips, at one-third the expense. At the deeper depressions, the bituminous patches pushed out into waves during the summer.

A 2-in. depression at Station 1 + 73 was cut out and filled with 3-in. creosoted wood blocks with asphalt-filled joints. The wood blocks at this point, along the trolley track, and in the two-course expansion joint at Station 0 + 97, have given good satisfaction, but they are expensive.

A traffic census taken May 18th, 19th, and 20th, 1912, from 7 A. M. to 11 P. M., showed an average daily traffic of 3 330 vehicles, of which 90% was motor traffic.

The speaker has inspected a number of concrete pavements between Boston and Chicago and has not seen any which have been down for

any length of time and are satisfactory for heavy traffic. Some of the Blome pavements in Chicago, Washington, and New Haven, have cracked badly. Most of the concrete pavements in New England have cracked and worn in holes, and have been coated with tar and grit. Mr. Thomes.

A concrete pavement is defined by some engineers as one having a cement-concrete wearing surface, and by others as concrete with a thin bituminous wearing surface. The type referred to under discussion should be clearly stated.

There may be places and conditions where each type is advisable, but it is wise to proceed cautiously with the ordinary class of concrete pavement, especially in the climate of New York. A good, rich, one-layer, concrete pavement, uniformly constructed with a smooth surface and covered with a thin bituminous mat, seems to have some advantages. A good example of this type was constructed in the summer of 1912 by the Hassam Paving Company on the Jericho Turnpike, on Long Island, about one mile east of the New York City limits.

Expansion joints are not advisable in grouted stone block or wood block pavements in which proper longitudinal expansion joints are provided, and it would seem that the same should apply to concrete pavements.

PHILIP P. SHARPLES, ESQ.*—In the application of refined tars on old concrete pavements, the question of adherence is very much harder to solve than in the case of new pavements. On many pavements where there is a great deal of dust, both brought on and due to the actual grinding up of the concrete, it is extremely difficult to obtain a clean surface on which to put the tar. Mr. Sharples.

If the pavement is dampened before the application of the refined tar, it is very important that absolutely all traffic be excluded until there has been a chance for the moisture to dry out from under the surface of the tar. Otherwise the tar does not adhere well, loosens, and will not give good results. In dampening the pavement no free water should be left on the top of the concrete, for in that case, the water tends to float the tar, preventing a good bond. The concrete should be allowed to dry partly, be squeegeed off, or treated in some other way so that no excess of moisture remains.

The tar should be applied in two thin coats. A slight coating of sand or screenings over the first coat is necessary, in order to allow the second coat to be applied. The second coat is filled with sand, screenings or pea stone. In this way a surface about $\frac{3}{8}$ or $\frac{1}{2}$ in. thick is built on top of the concrete. To insure a good wearing surface, it is essential to embed in the tar a hard mineral matter which will resist abrasion.

In repairing old concrete pavements, pot-holes must be considered. It is absolutely impossible to repair pot-holes with cement-concrete in an economical way. It has been found satisfactory, however, to use

* Chief Chemist, Barrett Mfg. Co., Boston, Mass.

Mr. Sharples. tar concrete, but, in doing so, it is important to use stone which will be practically as large as the hole is deep. If fine material is used, the traffic will soon force the tar mixture out of the hole.

Mr. Sibley. L. P. SIBLEY, ESQ.*—It has been suggested that the concrete should be moistened immediately before the application of the bituminous material. It seems to the speaker that there is a great difference between the entire body of the concrete being damp, or dry, and simply sprinkling the concrete immediately before applying the bituminous material. If the entire body of concrete is damp, it seems as if the results must be at least uncertain; but if it is dry, the moisture caused by the light quantity of water sprinkled on the surface would be sufficiently absorbed by the concrete so as not to interfere with the bond between the latter and the bituminous material. The road should be kept closed to traffic for a few hours in order to permit of this absorption.

In the case of bonding between two layers of tar, when the first layer has been covered with screenings of sand, it is very important that these screenings be free from dust or any fine material, no particle of which should be less than $\frac{1}{8}$ in. in size. When the bond is thus constructed, there will be no blanket of dust or fine material over the first layer of tar to prevent the second layer from uniting solidly with it.

Mr. Whinery. SAMUEL WHINERY, M. AM. SOC. C. E.—In the speaker's opinion, every concrete roadway pavement should be laid in two courses—a foundation and a surface course. The same functions are required as in other roadway pavements, that is, strength to support loads, and density and hardness to resist surface abrasion.

The surface course of a concrete pavement should be compounded and laid with something like the same care, skill, and thoroughness as the wearing course of an asphalt pavement. As a rule, this has not been done. Where the second or top course has been used, it has been laid more or less carelessly, with insufficient regard to quality of materials, uniformity of consistency, and thorough work.

Concrete pavements are not suitable for streets of heavy travel. They should never be used where a foundation of 4 in. of ordinary concrete and a surface course of 2 in. of special surface mixture, will not be ample to carry the loads and stand the abrasion of travel. There are, in every city, a large number of streets on which a properly constructed concrete pavement will give entire satisfaction. On such streets it will prove durable, dustless, possess most of the desirable qualities of a first-class roadway pavement, and give a larger return per dollar invested than any other kind of pavement.

The speaker agrees with Mr. Thompson in regard to the necessity of using only the best materials, and of giving special attention to the

* Asst. Eastern Mgr., Barrett Mfg. Co., New York City.

quality of the sand, which should be thoroughly tested. The hardness and toughness of the stone used in the top course is also an important consideration. The speaker specifies that the size of the stone shall range from that which will pass a screen of 1½-in. mesh to that which will be held on a screen of ½-in. mesh. The fine material and dust is objectionable, because of its tendency to segregate in the mass and because its varying quantity is likely to prevent that uniformity of mixture which is highly desirable. He requires that the sand shall be screened, that it shall be of grains of such size that 75% of the whole will fail to pass the No. 30 sieve, and that it shall be tested with regard to the strength of mortar made with it. This is important in view of recent unsatisfactory experiences with apparently good sand. A ratio of 1 part of cement to 2 parts of sand for the mortar is probably rich enough. In some recent specifications the speaker has called for ratios of 1:3:6 for the foundation course and 1:2:3.75 for the surface course, but in the compacted mass the volume of mortar should exceed that of the voids in the stone by from 7 to 10 per cent. The ideal surface-course mixture is one in which the largest practicable ratio of the mass is stone with sufficient strong mortar to hold the fragments firmly in place. The speaker tries to insist that this surface mixture shall possess great uniformity in composition and consistency. This requires: careful measurement of all the materials, including the water; that special care be taken to prevent segregation in the mixer or on the street; and that it shall be properly and uniformly graded and compressed to a true surface on the street; in short, that especial care shall be taken to make a concrete as nearly perfect as possible. Mr. Whinery.

It is very important that the surface course be laid and tamped on the foundation course before either begins to set. This requires that the construction of the two courses be carried on simultaneously.

The surface course should be finished with a roller, not only to perfect the consolidation, but to secure a true surface, which latter is nearly as important as in the case of asphalt pavement.

Finishing the surface with neat mortar or by troweling should be prohibited.

Our knowledge relative to expansion and cracking is as yet meager. Concrete pavements often crack where apparently ample expansion joints have been provided, and frequently where least expected. Where these natural cracks or joints occur, the wear along their edges, as a rule, is less than along those of artificial expansion joints. This is probably due to the fact that the wood or metal forms used for making the joints prevent the same uniformity of composition and density of the concrete along them as in the body of the work. To prevent this, it is better not to use such joint forms, but to cut the expansion joints through the concrete after it is laid. It is an open question at present whether it is not better to omit all expansion joints (except along the

Mr. Whinery.

curb) when the pavement is laid, and to let the concrete select and make its own expansion (or rather contraction) joints. These cracks should then be filled with bituminous cement, after being cleaned out as well as possible with proper tools and by water jets from high-pressure hose.

The question of the advisability of applying a coating of asphaltic oil to the surface of the pavement, is not yet settled, but there is every reason to believe that, if suitable material is used and the work is properly done, it will be decidedly advantageous.

Mr. Parker.

HAROLD PARKER, M. AM. SOC. C. E.—A concrete pavement is most desirable, provided it is properly laid; but, to obtain a uniform mixture and apply it uniformly all over the road is a problem not easy of accomplishment.

To secure a uniform concrete mixture on a road, the stone should be of absolutely uniform thickness, and rolled down so that the road is finished, as to cross-section and quantity of stone, before any cement is added. Whether mixtures are made on a board or in a machine, the very fact that the concrete has to be transported and put on the road in batches, destroys its uniformity, because of the difference in the specific gravity of the materials used. A perfectly uniform road, therefore, is impossible. This can be remedied in a measure by the care that is used in making the mixture and putting it down, but the only reliable concrete road is made, as the speaker has stated, by placing the stone without binder on the road bed and rolling it thoroughly before the cement is applied.

A properly constructed concrete road is the most indestructible surface that can be built except a stone pavement. Wherever the traffic is sufficiently heavy, or of a character to warrant it, a concrete foundation should be built for any surface.

The bituminous surface on a concrete road should be applied in very thin layers, as such a coating can be maintained at small expense. It can be put on under pressure, which gives a perfectly uniform quantity of bituminous material at very trifling expense. A defect which occurs in a road thus treated is so slight in elevation that it will not be felt by the ordinary traffic going over it, whereas a defect in a top surface of 2 in. or more, soon becomes a material one, which every vehicle feels. It also increases very fast, and disintegration once begun continues rapidly.

Mr. Kinney.

WILLIAM M. KINNEY, JUN. AM. SOC. C. E.—From the examination of more than 100 concrete pavements, during and after construction, in various sections of New York, Pennsylvania, Maryland, and other States, the speaker has come to the conclusion that the use of bank-run material or crusher-run limestone is almost sure to result in failure. The aggregates should be screened dry over a No. 4 screen, and remixed in the definite proportions desired. A mixture of 1 part of

cement, 2 parts of sand, and 3 parts of gravel or crushed stone (from ¼ in. to 1 in. in size), will give the best results under general conditions. Though a 1:2:4 mixture might be considered better to fill the usual requirement of the ratio of fine to coarse aggregate, the excess mortar in a 1:2:3 mixture will be a distinct advantage in finishing, and, under most conditions, will be satisfactory. Mr. Kinney.

This, of course, presupposes a one-course pavement, which should be selected in all cases where the aggregate available, in the territory in which the pavement is to be constructed, is suitable for a wearing surface. Good practice would recommend the two-course pavement only in places where local aggregate was suitable for the base concrete, but not for the wearing course. In such cases the cost of the pavement might be reduced considerably by shipping in only enough of the more satisfactory aggregate for the wearing course.

The use of steel trowels on a concrete pavement should be prohibited. The wooden float used after the concrete has been struck off to the proper grade with a straight-edge is amply sufficient, and does not draw the fine material to the surface like the steel trowel. The presence of fine material on the surface is objectionable from the wearing standpoint, and also because of increased slipperiness.

The protection of the concrete from premature drying out is an important feature of work of this type. As soon as the concrete becomes sufficiently firm, a canvas cover should be spread over the finished pavement and kept moist. This cover may be removed the next morning, or earlier, if the concrete is sufficiently hard, and the pavement covered with at least 2 in. of sand or earth, which should be kept sprinkled. The wearing quality of a pavement, well laid and of good materials, is often greatly reduced by failure to appreciate this one point of proper protection of the hardening concrete.

Regarding bituminous coatings for concrete pavements, there is this to be said in general: The coating will wear off sooner or later, and conditions in the United States are such that we cannot hope for the immediate replacement of this coating. It is essential, therefore, that the concrete be laid with the same care and attention as if no surfacing were to be applied. There is no hope for a road consisting of 6 or 7 in. of poor concrete and ½ in. of tar or asphalt sprinkled with sand. The question of the application of bituminous surfacing should be considered carefully, as it is certain that methods of application which work successfully with tars do not work well with asphalts, and *vice versa.*

JAMES OWEN, M. AM. SOC. C. E.—In building the Plank Road from Newark to Jersey City, N. J., on the salt meadows which had been filled up to reduce the vibration, it became necessary to provide a roadway during a certain portion of the winter of 1911-12. Mr. Owen.

Mr. Owen. As a matter of economy an 8-in. concrete pavement was laid, with the idea of allowing the traffic on it during the winter and then using it in the spring for a foundation for a permanent pavement. It was about 8 in. thick and about 20 ft. wide on the fills. After the road was partly finished on top, traffic was allowed on it, with the result that the pavement was ground up and went to pieces in about three weeks.

At that time the failure was attributed to some details of the construction and to the very heavy travel. During the summer of 1912 a granite block pavement was laid on one side, with cement grouted joints, under the usual specifications. The grouting went to pieces, although the travel was not allowed on it for from 10 to 20 days after it had been applied. It was suggested that the vibration of the road was the cause of the failure of both sections. It is evident that, in cases where there is a possibility of vibration, the bond of the cement will be impaired and the efficacy of the work destroyed.

(2) COST RECORDS AND REPORTS.

By Messrs. Nelson P. Lewis, L. L. Tribus, W. W. Crosby, George W. Tillson, William H. Connell, A. H. Blanchard, H. W. Durham, and Frederick Wilcock.

Nelson P. Lewis, M. Am. Soc. C. E.—The subject of cost records and reports may be considered from two aspects, the professional and the scientific. Mr. Lewis.

There can be no doubt of the right of the professional man to profit by discoveries and inventions resulting from his own experiments and researches, provided he does not in a professional capacity advise the use by his clients of appliances or processes on which he holds patents and receives royalties. The idea, however, that he may regard the results of his observations and the lessons to be drawn from his own successes and failures as his stock in trade is repugnant to all who appreciate the dignity of professional work and the necessity of high professional ideals. This is especially the case with one who occupies a public office and whose experiments are conducted with public funds. It is true that while some are able to profit by the mistakes of others, there are, perhaps, more who must make the same mistakes themselves before they can learn to avoid them, and, unfortunately, there are still others who appear incapable of learning much either from the mistakes of others or from their own. That is no reason, however, why they should not be given the opportunity to do so. Believing that the professional obligation of the engineer to make public the results of his own experience for the benefit of others will be conceded, this aspect of the subject will be dismissed without further comment.

Our chief interest, then, so far as this discussion is concerned, is the scientific value of cost records and reports. If they are unscientific and inaccurate, they are worse than useless; they are misleading, not only to those who examine them, but also to those who prepare them. They may be accurate as far as they go, and still be unscientific and all the more dangerous because they bear on their face certain evidences of painstaking accuracy, while, owing to their failure to give the precise circumstances under which the work was carried out, the omission of overhead or other charges, and especially owing to a disposition to ignore certain unsatisfactory features of the completed work due to carelessness or lack of foresight, which the engineer in responsible charge is reluctant to admit, erroneous conclusions are likely to be drawn.

The aim of the speaker in these introductory remarks will be, not to suggest certain forms of cost records and reports, but to outline

Mr. Lewis.

briefly some of the fundamental considerations which should govern their preparation and publication. First, we must draw a sharp distinction between cost records and accounting—two very different things. The main object of accounting is to permit of intelligent bookkeeping, to distribute the different items of cost among various accounts or appropriations, to know about how much the work has cost, and to distribute that cost with a refinement and an apparent accuracy that will delight the heart of the accountant. If some items are found to have been charged up in a manner inconsistent with the principles laid down by men who have never done any real constructive work, there is ground for caustic criticism, an opportunity for which is often productive of holy joy, which, however, is judiciously expressed in terms of indignation.

The object of cost records, on the other hand, is to determine how much the work is costing, and to check or correct leaks here or there, which may be due to improper organization or to ill-adapted plant or equipment. Such defects will not only add to the cost of the work, but they are frequently of such a character as to impair its quality when finished. Of course, when an intelligent system of cost records is first introduced, it cannot be expected to have an immediate effect on the particular job to which it is first applied. It will, however, indicate where improvement can be effected, and it should not be long before the system can be perfected so that the engineer can put his finger on the weak spots very quickly and correct the trouble before the record for the entire job shall have been spoiled.

Road construction and maintenance are usually paid for from public funds, and the engineer in charge of such work is almost invariably hedged about with legal restrictions and limitations, which have been placed on his authority, particularly as to the control of labor and his inability to buy in the cheapest market or to secure needed materials or supplies promptly in an emergency. It is not unnatural, under these circumstances, that he should come to regard cost records as of academic, rather than of practical, interest and value.

In the speaker's opinion, there is nothing that will operate more effectually to remove, or at least mitigate, these restrictions and limitations than an accurate, scientific presentation of the facts, with the reasons why. Even in the public service, it will be possible to select a certain number of skilled and common laborers who are fairly efficient, and if these can be grouped together and the results obtained with them compared with those of a gang of less industry and capacity, and the responsibility for low efficiency and greater cost clearly and publicly placed where it belongs, it is not unlikely that popular opinion will soon insist that the man who is responsible for results will be given a chance to produce them.

Efficiency is an impressive sort of word, and efficiency engineering sounds like a particularly dignified and useful branch of the Engineering Profession. It has naturally attracted a considerable number of men, many of whom have had little actual experience and whose capacity for destructive criticism is far more conspicuous than their ability to do constructive work. It would be a great mistake, however, to conclude that efforts to promote efficiency are not worth while. They are worth while, provided they result in a decrease of the actual unit cost of work and at the same time improve its quality. Unless this is the result, "the game is not worth the candle." If cost records do not point the way to economy without sacrifice of quality, they have only an academic value and are not worth while. If reports contain only a record of the number of contracts which have been let, the names of the contractors, and the amounts expended from each account, with the unexpended balances in each, they have no engineering value. If they are framed so as to make a favorable showing of the quantity of work done in a given time and a low cost per square yard or per mile, and at the same time omit data which might indicate that these results were accomplished at the sacrifice of quality, and if they lay emphasis on the successes and ignore the failures, they are dishonest. Mr. Lewis.

Cost records of road work should indicate as clearly as possible the proportion of the expense chargeable to:

1.—Permanent betterments, such as land purchases, grading, the improvement of lines and grades, masonry and steel bridges, and culverts. This part of the work once done may be considered permanent or sufficiently durable to outlast more than a single generation.

2.—Curbing, gutter pavement, fencing, guard-rails, wooden bridges, concrete foundation, planting, etc., all of which may need periodical repairs, but will probably last twenty years at least.

3.—The road surface, which will begin to deteriorate at once, and will need constant attention and periodical renewal.

For each piece of work the records should include:

1.—The character and first cost of the materials;

2.—Cost of delivery on the work, with kind of transportation and distance hauled;

3.—Cost of labor of all classes, and the quality of the same;

4.—Cost or present value of plant and equipment, with allowance for depreciation due to the work under construction and not previously marked off;

5.—All overhead charges, including engineering and inspection;

6.—The cost of bonds, permits, etc.;

7.—All delays due to weather, failure to receive materials, strikes, or other causes;

Mr. Lewis.

8.—A precise description of the methods used and of the surface treatment of the road;

9.—A statement as to the results obtained, the probable causes of failures or unsatisfactory work, and the means used to correct or remedy them;

10.—The manner in which the funds to pay for the work are raised, whether by cash appropriations, by the issue of bonds, with length of term and rate of interest, or by money advanced in anticipation of the collection of assessments for benefit.

Although this brief outline may have contributed nothing of value, it is hoped that it will serve the purpose of an introduction.

Mr. Tribus.

L. L. Tribus, M. Am. Soc. C. E.—The value of cost records is well illustrated in the Borough of Richmond, City of New York. In organizing its work the following queries were considered:

1. What is really needed to be done?
2. Was similar work done in preceding years necessary?
3. What had such work cost locally and in other places?
4. What should it cost?
5. How can appropriations and expenditures be controlled, so as to secure work needed and at proper cost?

Some few years ago, the City of New York as a whole felt that, in connection with its highways and sewers, there was a lack of reliable information on which the appropriating bodies could act. From its cost records, maintained for years, and practically the only ones in the city, the Borough of Richmond furnished most of the information from which appropriations were made thereafter for work throughout the Greater City.

In carrying out the plans, if a certain class of work costs less in one district than in another, an investigation is made. If the difference is large, there is some special reason for it, and the records usually show what that reason is, because conditions as to haul, temperature, and a number of other items which have an influence in the destruction or preservation of roads and sewers and in their maintenance, are known.

The speaker can testify that cost records and intelligent reports, with the giving of needed information to the men, produce results of economic value, fully within the term, "efficiency," as defined by Mr. Lewis.

Mr. Crosby.

W. W. Crosby, M. Am. Soc. C. E.—With the growing demand for efficiency in all engineering work, with the increase in attention generally to this feature, and with the recognition of the dependence on the factor of cost or expense for measuring the value of the results, it seems to have become necessary to regulate somewhat the acquisition and reporting of cost data. Up to the present, not only have

the costs of details of road work been almost as variously reported as there were reporters or details, but a similar variation has also existed in the statements of costs of the work in the aggregate or as a whole. Mr. Crosby.

For instance, if one desires to know what the total expense per mile has been to the State of Massachusetts for its completed State roads, and how that same cost unit of New York compares with the Massachusetts figures, no satisfactory, or even approximately correct, answer can be had, because of the different systems used in accounting for the road funds of the two States. The speaker had occasion, a year or more ago, to compare the cost per mile of the preliminary road surveys and plans in Maryland with those of some other State doing similar work, but he was able to find only two instances in the country where even approximate costs of this item could be had, one of these being a "post-mortem" by a special committee engaged for the purpose.

Take the simple case of a single item of road work—that of water-bound macadam. Speakers here and reporters elsewhere have variously calculated it, in some cases naming simply the price paid to a contractor; in others, where the work was done by "force account" and the machinery was rented, the rental actually paid for the machinery was included, but no depreciation, profit, or overhead charges were embraced in the figures; and, in still other instances, where perhaps the machinery was owned by the authorities, no allowances for it, nor even for the supervision, were calculated in the costs given. Yet the effort was, and is frequently, made to draw close comparisons between the resulting "cost" figures obtained by each method.

The speaker took occasion two years ago to invite attention to the importance of this matter, because of the readily apparent interest manifested in the statements of costs made by the various speakers. Further, it will probably be admitted that the satisfactory selection of a pavement to meet any local conditions will be largely affected, if not determined, by proper consideration of the cost factors. Hence the importance of accuracy in arriving at the arithmetical equivalent of these factors in the equation; and the narrower the margin, the greater the necessity becomes.

To avoid being misunderstood and considered perhaps as a "hair splitter," allow the speaker to state that exactness in this matter is not his goal, but rather "relative" accuracy. That is, he believes that almost all values are relative; that, in road matters, their dependence on other factors, such, for instance, as traffic, weather, and other conditions—impracticable of either exact determination or expression—relieves the necessity of absolute exactness, and that ordinary accuracy and uniformity are all that are necessary or desirable for purposes of satisfactory comparison, which is the end desired.

Mr. Crosby.

Therefore, in the interest of uniformity and for general benefit, attention is invited to the system for the accounting of road expenditures established in Maryland in the spring of 1912 by the speaker with the assistance of a reputable firm of expert accountants (Haskins and Sells, of New York). Although complete information in the matter could not be appropriately given on this occasion, the principles involved may be indicated briefly. They are:

First.—Charge to any item all the expenses that can be accurately determined as belonging properly to that item.

Second.—Charge to any item the cost of tools, materials, machinery, etc., consumed by that item. In the cases of tools or machinery having a life greater than their period of use for any item, then charge to that item, in lieu of depreciation, rent, interest, etc., a flat rate of 5% per month (or if preferable for any reason, 0.2% per day) for the time used on the item in question.

Third.—Such expenses as are unassignable clearly to any items, or indivisible among a group of items, are to be pro-rated among such items or groups in the proportion that the individual totals of the costs directly chargeable to such items or groups bear to the aggregate of such direct charges.

These principles can be carried into the details of the work as far as desired. If carefully applied, they seem to permit of sufficiently accurate results to enable comparisons of cost to be made intelligently, a thing now most difficult, if not impossible, between different localities not under identical authorities.

Two of the principles may seem to be simple and not new; but the speaker has found only one or two instances of even an approach to uniformity in their application, though he has searched far and wide.

The second principle, regarding charges for machinery, is probably the most novel of the three. It will not be disputed that a charge should be made for tools and machinery, but what this charge should be, and how it should be applied, may cause some discussion. The rate suggested by the speaker corresponds fairly closely with prices secured by private owners; it is simple and convenient, and to the speaker does not seem to be unfair. He admits that it will probably seem too high at first, but after experience with the relatively short life of road machinery, the desirability of keeping it in first-class condition, the length of time it is frequently idle, consideration of interest charges, obsolescence, etc., he thinks the rate suggested is wise and safe. It should be stated, perhaps, that this 5% per month is in addition to bills for minor repairs which are chargeable to the work directly.

In illustration of the application of the foregoing, the speaker submits two graphical representations of the subdivision of cost or expense into items of ordinary use—Tables 1 and 2. Table 2 shows the figures and percentages for each item or group of items in relation to the total of each group, and of the whole expense of the State Roads Commission of Maryland for 4 years (1908-'09, '10-'11), aggregating $4 250 000. Mr. Crosby.

He also submits a subdivision (Table 3), on the same basis, of the part of this expense for completed work, showing its division between the physical items ordinarily reported by the various authorities, and of general public interest.

TABLE 1.

Cost.	Administration and Legal........	Commission's (or Board's) Salaries and Expenses. Commission's (or Board's) Office Force, Salaries. Commission's (or Board's) Office Expenses. Commission's (or Board's) Counsel, Salaries, Fees and Expenses.
	Engineering, General............	Chief Engineer's Salary and Expenses. Chief Engineer's Office Employees' Salaries. Chief Engineer's Office Expenses. Chief Engineer's Instruments and Repairs. Chief Engineer's Laboratory Investigations.
	Preliminary........................	Engineers in Charge—Salary and Expenses. Survey Parties—Salary and Expenses. Office Employees' (Draftsmen, etc.) Salaries. Office Expenses, Stakes, etc.
	Construction. (Reconstruction) (Maintenance)	Engineers in Charge—Salaries and Expenses. Office Employees—Salaries. Office Expenses. Inspectors—Salaries and Expenses. Superintendents—Salaries and Expenses. Foremen, Labor, Teams, etc., Pay of Advertising. Materials Consumed, Expense for. Use of Equipment, Rental for and Repairs. Rights of Way and Damages. Advertising. Miscellaneous.
	Equipment..........................	Expense for. Transportation of and Establishment. Renewals and Depreciation. Salaries of Men in Charge of, etc.

In reference to this, let it be said that it was fully realized that seldom, if ever, are two appropriations made along identical lines, and that there will always be a necessity for accounting for the funds separately. Provision has been made for this matter, also, in the new Maryland system, and those interested in the bookkeeping are referred to the recent Report of the Roads Commission for further information on this point.

The foregoing is applicable to the records and compilations of the first cost of road work. As to the final cost of any particular piece of work, it is evident that such is the sum of the first cost and the maintenance and interest charges, or the "long-run cost."

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TABLE 2.

				Per-cent-age.	Amount.
Total Expense $4 422 520.54.	Administration and Legal $62 910.51 1.4 per cent.		Commission Salaries and Expenses	50.33	$ 31 666.42
			Commission Secretary and Office Employees' Salaries	23.41	14 728.05
			Commission Office Expenses	16.33	10 273.71
			Counsel's Salary, Fees and Expenses	9.93	6 242.33
	Engineering General $56 318.72 1.3 per cent.		Engineer's Salary and Expenses	27.81	15 660.59
			Office Employees' Salaries	28.62	16 118.02
			Office Expenses	37.49	21 116.22
			Shop Labor and Material	5.01	2 820.29
			Tests and Investigations	1.07	603.60
	Preliminary and Construction $4 126 911.46 93.3 per cent.	Engineering $56 056.90 1.3 per cent.	Engineer's Salary and Expenses	27.96	15 675.81
			Engineer Inspector's Salary and Expenses	38.53	21 598.72
			Office Employees' Salaries	31.54	17 680.85
			Office Expenses	1.97	1 101.52
		Preliminary $59 907.12 1.5 per cent.	Survey Parties	34.06	20 405.63
			Draftsmen	65.94	39 501.49
		Construction $4 010 947.44 97.2 per cent.	Rights of Way and Damages	6.91	277 157.76
			Grading	15.45	619 652.83
			Surfacing	56.10	2 250 318.16
			Bridges and Culverts	12.65	507 214.33
			Drains	1.20	48 103.13
			Advertising	0.07	2 976.87
			Inspection	2.44	97 677.18
			Superintendence	0.40	16 242.81
			Final Surveys, Estimates and Plans	0.03	1 100.00
			Miscellaneous	0.37	14 789.26
			United Railways Company's Work	4.38	175 705.11
	Reconstruction and Maintenance $128 949.47 2.9 per cent.	Engineering $14 293.25 11.1 per cent.	Engineers' Salary and Expenses	19.07	2 725.21
			Engineer Inspector's Salary and Expenses	50.82	7 263.66
			Office Employees' Salaries	23.93	3 421.05
			Office Expenses	6.18	883.33
		Reconstruction $16 378.92 12.7 per cent.	Labor and Materials	33.65	5 512.12
			Team Hire and Use of Equipment	21.46	3 515.02
			Superintendence	32.20	5 274.14
			Inspection	12.69	2 077.64
		Maintenance $98 277.30 76.2 per cent.	Labor	22.20	21 820.41
			Materials	57.31	56 320.75
			Team Hire and Use of Equipment	13.66	13 419.10
			Superintendence	5.77	5 675.98
			Inspection	1.06	1 041.06
	Equipment $47 430.38 1.1 per cent.	Construction	Rental charge for average time at 5% per month equals		14 563.00
		Maintenance	Rental charge for average time at 5% per month equals		595.00
		Reconstruction	Rental charge for average time at 5% per month equals		800.00

NOTE.—The figures in this table are given to show the actual distribution of the total expenditures by the State Roads Commission to December 31st, 1911. The segregation into items of work done is given on pages 28, 29, 30, and 31 of the report.

The "long-run cost" the speaker conceives is made up thus: Mr. Crosby.

Long-run cost:
- Interest on first cost. (When first cost is met by bonds.)
- Sinking fund charges.
- Maintenance expenses (made up periodically along lines prescribed above for first costs).
  on:
  - Surfacing,
  - Earthwork,
  - Culverts, bridges, and drains separately, and
  - Total.

Consequently, for recording the long-run costs in Maryland, cards were designed as shown by Table 4.

Lest it be thought that the speaker has, by his suggestions, simply added work and expense to the State highway authorities for the sole purpose of furnishing engineers with more accurate data, perhaps to an inappreciably good end, he may state that, on the contrary, the revision of the accounting system of the Maryland Roads Commission has resulted in at least two direct benefits:

First.—The accounts now kept give dependable information on all points likely to arise, and give it clearly, readily, and accurately; and it has been possible to secure easily and quickly from these accounts such information on other less usual points.

Second.—The annual expense of keeping the books of the Maryland Commission has been reduced by 60% or more.

The speaker believes that the Maryland system as adopted is readily applicable to other communities, and, when thus applied, would generally be found to be beneficial. The particular point he wishes to make is that far greater uniformity than at present exists in cost recording and reporting is now, not only desirable, but necessary, if as engineers we are to be able to do our full duty.

Connected with the foregoing, concerning cost figures, the speaker has some records of expenditures and of traffic on a particular road, which records, together with some deductions or conclusions of his own, he submits, as follows:

The experimental pitch-macadam pavement on Park Heights Avenue, near Baltimore, Md., is now entering its fourth year of service. Some of the sections were completed during the summer and autumn of 1909 (Sections 1 to 9 and 19 to 27, inclusive), and others during the working season of 1910. It is thought that sufficient experience has been had with most of these sections to warrant some study of the costs, and that such a study will be profitable.

Full descriptions of the construction have already been published,* and, therefore, need not be repeated here.

* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 74; and Vol. LXXV, p. 598.

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TABLE 3.—SUBDIVISION OF EXPENDITURES BY THE STATE IN THE YEARS 1908

County.	Contract No.	Name of road.	Description.	Miles of road.	Preliminary surveys and plans.	CONSTRUC- Grading.	Surfacing.	Bridges and culverts.	Under-drains.
......	....	....	"State Roads".......	153.13	$11 651.93	$269 407.14	$1 042 058.56	$139 205.94	$27 432.64
......	....	....	"State Aid Roads"...	26.89	2 015.15	59 656.02	164 107.58	49 320.26	3 786.70
......	....	....	"Roads and Bridges"	13.28	1 607.62	23 234.62	123 377.54	12 446.72	1 502.28
......	....	....	"State Road No. 1"..	0.55	123.63	903.53	20 928.11	287.04	482.31
Totals	....	....	All kinds............	193.85	$15 398.33	$353 201.31	$1 350 471.79	$201 259.96	$33 203.93

The expenditures on the twenty-six sections of pitch-macadam, on the two surfaces treated with pitch, and on the single section treated with "Glutrin," are given in Table 5. The travel censuses, showing the character and extent of the traffic on the road, are given in Table 6.*

On account of the rebuilding of Sections 1 and 3 late in 1911 (completed early in 1912), it is thought best to omit these two sections from detailed consideration now, simply remarking that at present they are in fair condition, their defects seeming to be due, mainly, to a slight excess of pitch in spots, probably from the use of too much material as a flush coat. Section 11-A may also be omitted as of little value for deductions, as it was an emergency section pitched with an unrecorded mixture of a variety of pitches happening to be available under the exigencies of the occasion.

Let it be assumed that, in the consideration of the expenditures for the pitch work, the expenses for resurfacing the old macadam, or for putting it in condition to receive the pitch treatment, do not enter; that the unrecorded (and difficult, if not impossible, to estimate) saving from not water-binding this macadam resurfacing can be neglected; and that the maintenance expenditures on the shoulders, earth side-road, and ditches should be eliminated where separable. Then Table 7 shows the net charges on the pitch-macadam.

The charges do not include any allowances for "Overhead Expenses," "Administration," or "Engineering," but cover all expenses for materials, labor, inspection, tools, etc., on the work, rental (or allowance therefor) on machinery used, etc., and in make-up follow the lines adopted for accounting by the State Roads Commission of Maryland.†

* An explanation of the "Factors" used may be found in "A Simplification of Traffic Censuses for Purposes of Comparison." *The Surveyor* (London), March 10th, 1911, p. 364: or *Municipal Engineering*, March, 1911, p. 167.

† First Report, State Roads Commissioner of Maryland for 1908-09-10-11.

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Roads Commission of Maryland for Completed Roads to 1912, Inclusive.

TION.				Rights of way and damages.	Total, including rights of way and damages.	Administration, legal and general engineering expenses.	Total cost of road.	Cost per mile.
Inspection and superintendence.	Miscellaneous.	Total.	Cost per mile.					
$37 538.25	$7 321.39	$1 534 615.85	$10 021.65	$3 032.09	$1 537 647.94	$70 088.24	$1 607 736.18	$10 499.16
11 841.38	1 235.51	291 962.60	10 857.67	......	291 962.60	13 189.89	305 152.49	11 348.17
2 124.90	212.56	164 506.24	12 387.52	1 915.85	166 422.09	7 582.93	174 005.02	13 102.79
736.72	51.48	23 512.82	42 750.58	......	23 512.82	1 077.08	24 589.90	44 708.91
$52 241.25	$8 820.94	$2 014 597.51	$10 387.40	$4 947.94	$2 019 545.45	$91 938.14	$2 111 483.59	$10 892.36

The following conclusions are drawn from these data and the speaker's experience with the work and its results:

Section 29, having been newly resurfaced with a fairly hard limestone water-bound macadam in 1909, and having shown by the summer of 1910 a tendency to serious deterioration under the travel on it, and since the latter date having been treated (as apparently necessary) with "Glutrin," and now being in fully as good condition as before the treatment was begun, the speaker is of the opinion that:

(*a*) A 12-ft., water-bound, limestone macadam will not support successfully travel aggregating 400 units per average day of 10 hours, and where 60% of the total travel is that of motors.

(*b*) Such a road surface can be made to sustain such travel, with physical satisfaction, by treating it with "Glutrin."

(*c*) Such a road surface can be made to sustain such travel, with physical satisfaction, by giving it a surface treatment of "cut-back Texas asphalt" (Section 28).

(*d*) An 18-ft., water-bound, trap rock macadam is incapable of sustaining, without serious deterioration in a few months, travel aggregating 2 500 units per average day of 10 hours, and where 90% of the total travel is that of motors.

(*e*) The surface treatment of such a road with a proper cut-back sludge asphalt, or a proper compound of water-gas tar, will enable it to sustain such travel, with physical satisfaction (Sections 15 and 16).

(*f*) A 24-ft. pitch-macadam, of either limestone or trap rock, will sustain, with at least physical satisfaction, travel aggregating as many as 15 000 units per average day of 10 hours, and when more than 95% of the total travel is that of motors (Sections 4 and 5).

(*g*) A 14-ft. pitch-macadam, of either limestone or trap rock, will sustain, with at least physical satisfaction, travel aggregating 5 000

Mr. Crosby.

TABLE 4.—SUMMARY OF MAINTENANCE COSTS.*

Road, ..., Sec. ......, Built under Contract No......, in .... County.

Completed Costs: Surfacing (sq. yd.) = $. Earthwork and Miscellaneous = $.

Bridges, Culverts, and Drains = $

Costs.

Period.	Surfacing.			Earthwork.			Culverts, Bridges, and Drains.			Grand total.
	Interest on cost and S. F. charges.	Expense of repairs and maintenance	Total.	Interest on cost and S. F. charges.	Expense of repairs and maintenance.	Total.	Interest on cost and S. F. charges.	Expense of repairs and maintenance.	Total.	
Int. at 6% (simple)..										

*Form for recording on cards.

units per average day of 10 hours, and when less than 15% of the total travel is that of horse-drawn vehicles (Sections 17 to 27, inclusive). Mr. Crosby.

(*h*) The present indications are that the use of pitch in quantities of more than 1¾ gal. per sq. yd., in the construction of pitch-macadam, where the upper course of the macadam is to be 3 in. thick after compaction, is generally inadvisable.

(*i*) The use of a flush coat, except possibly in the cases of pitches having unusual stability under the application of heat, is inadvisable in the construction of pitch-macadam, unless the proportion of horse-drawn travel to motor travel is considerably higher than in this case.

(*j*) With the use of materials susceptible to heat, or fairly so—such as most tars—the application of the pitch to the rolled stone can safely be stopped at a line parallel to and 1 ft. within the edges of the stone, and reliance can be felt that this border of the macadam, later, will be properly bound by the flow of the pitch under heat and travel.

(*k*) With the use of asphaltic or blown-oil pitches, having a "penetration" at normal temperatures harder than 150, the application of the pitch should be made fully to the edges of the rolled stone.

(*l*) The requisite penetration of the pitch into the rolled stone is obtained when the pitch reaches to the bottom of the average sized stone in the course penetrated.

(*m*) Success in the construction of pitch-macadam depends primarily on the proper compaction of the macadam particles, and is directly proportional to the success in rolling the macadam before spreading the pitch.

(*n*) The use of sand on top of the pitch is open to grave objections, and the value of its use before spreading the pitch is questionable, clean stone chips of small gravel, free from particles which will not pass ⅛-in. mesh being preferable.

(*o*) With the use of a proper pitch, there is no material advantage in closing the finished pitch-macadam to travel for a period after its construction, provided the adhesion of tires to the surface is prevented by watering, spreading an excess of grit, or by other means.

(*p*) For pitch-macadam, a pitch, to be satisfactory, should possess at least the following characteristics:

(1) It should flow freely, though not excessively so, under the conditions of application.

(2) It should show, after evaporation at 105° cent. for 21 hours (20 grams in 3½-in. dish), a penetration of not less than 10 at 4° cent., and not more than 100 at 25° cent. (No. 2 needle, 100 grams, 5 sec.).

(3) The naphtha-soluble portions of the pitch, when evaporated on glass, should show themselves to be decidedly adhesive.

Mr. Crosby.

TABLE 5.—CONSTRUCTION AND MAINTENANCE

Section.	Material.	Width, in feet.	Area, in square yards.	Dates of use.	Gallons used per square yard.	Cost of resurfacing.
				CONSTRUCTION OF MACADAM PER		
1..	Texas	30, 35, 24	11 226.11	July, Aug., '09	2.39	$0.337
2..	Gulf Ref. Co. "Asphaltoil A"	24	1 936.00	Aug., Sept., '09	3.97	0.339
3..	Texas	24	1 242.66	Sept., 1909	3.22	0.336
4..	Imp. Prod. Co. "Fairfield"	24	1 173 33	Sept., 1909	3.67	0.337
5..	U. G. I. No. 4—Sample No. 1	24	1 712.00	Sept., 1909	3.12	0.339
6..	Warren-Puritan, Brand No. 10.	24	1 826.66	Sept., Oct., '09	4.19	0.333
7..	Tarvia X	24	1 909.33	October, 1909	5.40	0.337
8..	Amer. Tar Co., "Tarite"	24	1 816.00	October, 1909	4.41	0.336
9..	U. G. I.—(1909 work)	24	1 170.66	November, 1909	4.46	0.340
10..	U. G. I.—(1910 work)	24	3 232.00	May, June, '10	1.43	0.397
11..	Texas	24	5 981.33	June, 1910	1.25	0.397
11-A	Mixed—52 bbl	24	1 261.33	June, 1910	1.65	0.397
12..	Headley Man'f. Co	24	4 685.83	June, July, '10	1.70	0.397
13..	Barber Asphalt Co	24	4 202.66	July, Aug., '10	1.45	0.397
14..	"Fairfield"	24	3 184.00	Aug., Sept., '10	1.69	0.397
15..	"Fairfield Antidust"	18	5 808.00	October, 1910	0.61	0.397
16..	U. G. I. "Antidust"	18	1 100.00	Sept., Oct., '10	0.94	0.397
17..	"Sarco"	12	3 568.00	October, 1910	1.42	0.397
18..	Std. Oil (1910 work)	12	1 898.33	October, 1910	1.70	0.397
19..	Std. Oil (1909 work)	14	127.55	November, 1909	3.92	0.241
20..	U. G. I.	14	1 294.22	October, 1909	3.86	0.230
21..	Texas	14	693.77	October, 1909	4.70	0.228
22..	Gulf "Asphaltoil A"	14	976.88	October, 1909	2.95	0.229
23..	Warren-Puritan, Brand No. 17	14	844.66	October, 1909	4.71	0.224
24..	U. G. I.	14	2 246.22	September, 1009	2.23	0.229
25..	Imp. Prod. Co. "Fairfield"	14	1 532.22	September, 1909	2.73	0.228
26..	Con. G. E. L. & P. Co. (Complete top of U. G. I.)	14	3 758 22	August, 1909	2.98	0.216
27..	Con. G. E. L. & P. Co.	14	1 048.44	August, 1909	1.95	0.216
28..	"Texaco Special"	14	863.33	Oct., Nov., '10	1.25	0.607
29..	"Glutrin"	12	13 398.67	Nov., 1909; Oct., Nov., '10.	0.45	0.318

* Length, 8.246 + miles.

† Includes maintenance of shoulders, side-road, and drains or gutters.

‡ To remedy slipperiness.

(*q*) In the cases of tar-pitches or pitch compounds, and for the purpose of lessening their susceptibility to temperature changes occurring after use, it is desirable that "free carbon" (matter insoluble in carbon bisulphide) be present to an extent greater than a minimum percentage of the fractional distillate from such pitches between 225 and 300° cent.; but, in order to avoid excessive hardness at low temperatures, this percentage of "free carbon" should be less than a maximum—the minimum and maximum referred to being largely dependent, for their expression in figures, on the local conditions.

(*r*) A 12-ft. width for the road metal is not economical where the travel aggregates more than 400 units per average day of 10 hours; nor

EXPENDITURES ON PARK HEIGHTS AVENUE.*

Mr. Crosby.

Square Yard.		Maintenance, per Square Yard of Macadam.										
			1911.					1912.				
Cost of pitching, including shipping.	Total first cost.	1910.	Earthwork and culverts.†	Painting edges.	Oiling and chipping.‡	Net repairs.	Totals.	Earthwork and culverts.	Painting edges.	Net repairs.	Totals.
$0.327	$0.664	$0.088	....	....	....	$0.9011 §	$0.9011	$0.0396	....	$0.0036	$0.0432
0.434	0.773	0.080	$0.0455	....	....	0.0000	0.0455	0.0199	....	0.0959	0.1158
0.418	0.754	0.187	....	....	....	0.9947 §	0.9947	0.0124	....	0.0044	0.0168
0.449	0.786	0.081	0.0093	....	....	0.0000	0.0093	0.0065	....	0.0031	0.0096
0.344	0.683	0.081	0.0049	....	$0.0364	0.0093	0.0506	0.0169	....	0.0034	0.0203
0.606	0.939	0.079	0.0024	....	....	0.0563	0.0587	0 0183	....	0.0434	0.0617
0.618	0.955	0.082	0.0054	....	0.0180	0.0034	0.0268	0.0075	....	0.0135	0.0210
0.605	0.941	0.080	0.0097	....	....	0.0098 ‖	0.0195	0.0064	....	0.0583	0.0647
0.454	0.894	0.091	0.0187	....	....	0.0049	0.0236	0.0088	....	0.0171	0.0259
0.242	0.639	......	0.0097	....	....	0.0080	0.0177	0.0022	....	0.0052	0.0074
0.264	0.661	......	0.0028	....	....	0.0042	0.0070	0.0018	....	0.0039	0.0057
0.262	0.659	......	0.0055	....	....	0.0298	0.0353	0.0087	....	0.0100	0.0187
0.327	0.724	......	0.0029	....	....	0.0349	0.0378	0.0059	....	0.0005	0.0064
0.325	0.722	......	0.0057	$0.0101	....	0.0000	0.0158	0.0040	....	0.0005	0.0045
0.292	0.689	......	0.0048	0.0013	....	0.0105	0.0166	0.0099	....	0.0215	0.0314
0.084	0.481	......	0.0037	....	....	0.0335	0.0372	0.0025	....	0.0157	0.0182
0.140	0.537	......	0.0070	....	....	0.0980	0.1050	0.0045	....	0.0243	0.0287
0.325	0.722	......	0.0136	0.0161	....	0.0000	0.0297	0.0209	....	0.0143	0.0352
0.287	0.684	......	0.0091	0.0281	0.0062	0.0000	0.0434	0.0278	....	0.0072	0.0350
0.410	0.651	0.087	0.0266	....	0.0484	0.2179	0.2929	0.0055	....	0.0649	0.0704
0.408	0.638	0.088	0.0683	....	0.0332	0.0513	0.1528	0.0207	....	0.0000	0.0207
0.551	0.779	0.088	0.0186	....	....	0.1805	0.1991	0.0080	....	0.0239	0.0319
0.405	0.634	0.082	0.0391	....	....	0.1851	0.2242	0.0096	....	0.0670	0.0766
0.691	0.915	0.082	0.0144	....	0.0293	0.0463	0.0900	0.0116	....	0.0062	0.0178
0.257	0.486	0.081	0.0114	....	0.0041	0.0514 ¶	0.0669	0.0093	....	0.0088	0.0181
0.353	0.581	0.057	0.0091	....	....	0.0763 ¶	0.0854	0.0119	....	0.0000	0.0119
0.303	0.519	0.057	0.0038	....	....	0.0286 ¶	0.0324	0 0064	....	0.0024	0.0088
0.227	0.436	0.057	0.0086	0.0053	....	0.0003	0.0142	0.0065	....	0.0020	0.0085
0.355	0.962	......	0.0107	0.0068	....	0.0024	0.0199	0.0290	....	0.0442	0.0732
0.074	0.392	......	0.0138	....	....	0.0308	0.0446	0.0193	....	0.0874	0.1067

§ Reconstructed, 1911.
‖ Chips hauled and spread on account of bleeding.
¶ Rolling in No. 2 stone and chipping.

is the 14-ft. width economical where the travel exceeds 2 000 units and the proportion of motor travel is more than 80% of the total.

(*s*) Where high-speed motor travel forms a large proportion of the total, the unit width of travel lines should be considered as 9 or 10 ft., instead of 7 or 8 ft., as heretofore, because of the greater clearance required for the safe passing of the units of such travel.

Further economic conclusions seem to be rather premature at this time, and such, including a deduction concerning the possible advantage of using an excess of pitch during construction in order to prolong the life of the pitch-macadam, and perhaps in the end to reduce the cost, it would seem preferable to defer for two or three years.

Mr. Crosby.

TABLE 6.—TRAVEL CENSUS AND

Classification.	Maryland factor used in product.	Section 1. Units on dates named per period of 10 hours. Nov. 14, 1910.	Nov. 17, 1910.	Oct. 19, 1911.	May 4, 1912.	May 10, 1912.	May 11. 1912.
One-horse vehicles..........	2	376	442	438	366	608	514
Two " "	4	364	332	408	456	560	468
Three " "	6	48	48	....	30	60	48
Four " "	8	104	72	16	....	16	....
Six " "	12	24	....	12	....	....	....
Total horse traffic.......	..	916	894	874	1 052	1 244	1 030
MOTOR TRAFFIC:							
Motorcycles..............	2	14	14	50	102	96	136
Runabouts..............	10	200	230	480	590	450	830
4- or 5-seat cars..........	20	2 100	2 580	6 580	11 960	11 660	18 800
7-seat cars..............	40	1 920	2 480	5 320	9 920	5 120	9 760
Omnibuses or drays.......	20	40	80	440	700	760	980
Total traffic..............	..	5 190	6 278	13 744	24 324	19 330	41 536
Weather..................	..	Clear.	Clear.	Clear A. M. Rain P. M.	Clear.	Clear.	Clear.

TABLE 6.—

Classification.	Maryland factor used in product.	Section 2. Units on dates named per period of 10 hours. Nov. 15, 1910.	Nov. 18, 1910.	Oct. 14, 1911.	Oct. 21, 1911.	May 10, 1912.	May 11, 1912.
One-horse vehicles..........	2	336	314	422	314	632	636
Two- " "	4	356	312	376	248	512	476
Three- " "	6	18	18	....	....	30	90
Four- " "	8	152	96	16	8	16	....
Six- " "	12	....	24	....	....	....	....
Total horse traffic........	..	862	764	814	570	1 190	1 202
MOTOR TRAFFIC:							
Motorcycles..............	2	6	8	80	....	80	136
Runabouts..............	10	130	120	550	370	640	840
4- or 5-seat cars..........	20	2 200	2 500	8 080	5 120	9 300	15 440
7-seat cars..............	40	1 520	2 120	5 280	5 320	9 880	11 400
Omnibuses or drays.......	20	120	120	640	540	840	1 060
Total traffic..............	..	4 838	5 632	15 444	11 920	21 930	30 078
Weather..................	..	Cloudy.	Clear.	Clear.	Rain.	Clear.	Clear.

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UNITS ON PARK HEIGHTS AVENUE.

Sections 17 to 21. Units on dates named per period of 10 hours.					Section 28. Units per 10 hours.		
Nov. 22, 1910.	Nov. 26, 1910.	Oct. 25, 1911.	Apr. 1, 1912.	Apr. 16, 1912.	Jan. 16, 1911.	Apr. 10, 1912.	May 29, 1912.
44	40	44	444	176	28	28	28
44	28	24	748	372	4	8	12
....	....	6	....	....	....	....	....
....	8	8	272	152	40	24	9
36	24	....	336	120	132	96	1
124	100	82	1 800	820	204	156	50
4	8	16	80	26	....	6	....
40	20	230	680	440	....	30	1
560	760	960	4 960	3 120	....	160	19
480	760	2 680	2 400	1 160	....	....	5
60	40	60	1 060	420	....	40	5
1 268	1 688	4 028	10 980	5 986	204	392	80
Rain.	....	Clear.	Clear. Cloudy.	Clear.	Clear. Cold.	Clear.	Clear.

(*Continued.*)

Sections 15 and 16. Units on dates named per period of 10 hours.					Section 27. Units per 10 hours.		Section 29. Units per 10 hours.	
Nov. 16, 1910.	Nov. 19, 1910.	Nov. 21, 1910.	Nov. 25, 1910.	Apr. 12, 1912.	Nov. 23, 1910.	Nov. 28, 1910.	Nov. 24, 1910.	Nov. 29, 1910.
78	94	56	46	40	68	70	26	22
100	52	52	76	36	20	40	16	12
20	12	12	18	....	....	....	....	....
40	8	8	....	....	40	16	24	8
12	....	12	12	....	216	96	96	....
250	166	140	152	76	344	222	162	42
8	14	6	6	34	6	4	....	....
80	100	40	50	180	30	10	10	20
1 440	1 680	1 260	1 050	2 360	1 000	320	120	120
1 080	1 520	1 440	1 200	760	1 480	520	80	80
40	60	60	60	220	50	40	....	....
2 898	3 540	2 946	2 528	3 630	2 920	1 116	372	262
Clear.	Clear.	Cloudy.	Clear.	Clear.	Fair.	Cloudy. Rain.	Fair.	Cloudy.

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TABLE 7.—NET CHARGES FOR CONSTRUCTION AND MAINTENANCE OF PITCH-MACADAM ON PARK HEIGHTS AVENUE.

Section.	Width, in feet.	Pitching (first cost) per square yard.	MAINTENANCE OF MACADAM, PER SQUARE YARD. 1910	1911	1912	Total.	Average per year.	Yearly interest (6%) on first cost.	Yearly cost. (Average).	Average of traffic census, Maryland units.	Average yearly cost (per square yard) per traffic unit per foot of width, in mills.
1	24 to 35	$0.327	$0.088	$0.9011	$0.0036	$0.9927	(a)	......	......	......	......
2	24	0.434	0.080	0.0000	0.0959	0.1759	$0.0586	$0.0260	$0.0846	(b) 15 000	0.135
3	24	0.418	0.187	0.9947	0.0044	1.1861	(a)	......	......	......	......
4	24	0.449	0.081	0.0000	0.0031	0.0841	0.0280	0.0269	0.0549	9 600	0.137
5	24	0.344	0.081	0.0457	0.0034	0.1301	0.0434	0.0206	0.0640	9 600	0.160
6	24	0.606	0.079	0.0563	0.0434	0.1787	0.0596	0.0364	0.0960		0.320
7	24	0.618	0.082	0.0214	0.0135	0.1169	0.0390	0.0371	0.0761		0.254
8	24	0.605	0.080	0.0098	0.0583	0.1481	0.0494	0.0363	0.0857		0.286
9	24	0.454	0.091	0.0049	0.0171	0.1130	0.0377	0.0272	0.0649	7 200	0.216
10	24	0.242	.....	0.0080	0.0052	0.0132	0.0666	0.0145	0.0211		0.070
11	24	0.264	.....	0.0042	0.0039	0.0081	0.0041	0.0158	0.0199		0.066
11-A	24	0.262	.....	0.0298	0.0100	0.0398	0.0199	0.0157	0.0356		0.178
12	24	0.327	.....	0.0349	0.0005	0.0354	0.0177	0.0196	0.0373		0.187
13	24	0.325	.....	0.0101	0.0005	0.0106	0.0053	0.0195	0.0248	4 800	0.124
14	24	0.292	.....	0.0118	0.0215	0.0333	0.0167	0.0175	0.0342		0.171
15	18	0.084	.....	0.0335	0.0157	0.0492	0.0246	0.0050	0.0296		0.171
16	18	0.140	.....	0.0980	0.0243	0.1223	0.0612	0.0084	0.0696	(b) 3 114	0.402
17	12	0.325	.....	0.0161	0.0148	0.0304	0.0152	0.0195	0.0347		0.174
18	12	0.287	.....	0.0343	0.0072	0.0415	0.0208	0.0172	0.0380	2 400	0.190
19	14	0.470	0.087	0.2663	0.0649	0.4182	0.1394	0.0282	0.1676		1.117
20	14	0.408	0.088	0.0845	0.0000	0.1725	0.0575	0.0245	0.0820		0.547
21	14	0.551	0.088	0.1805	0.0239	0.2924	0.0975	0.0331	0.1306		0.871
22	14	0.405	0.082	0.1851	0.0670	0.3341	0.1114	0.0243	0.1357	2 100	0.905
23	14	0.691	0.082	0.0756	0.0062	0.1638	0.0546	0.0415	0.0961		0.641
24	14	0.257	0.081	0.0555	0.0088	0.1453	0.0484	0.0154	0.0638		0.425
25	14	0.353	0.057	0.0763	0.0000	0.1333	0.0444	0.0212	0.0656		0.437
26	14	0.303	0.057	0.0286	0.0024	0.0880	0.0293	0.0182	0.0475		0.317
27	14	0.227	0.057	0.0056	0.0020	0.0646	0.0215	0.0136	0.0351	(b) 2 030	0.242
28	14	0.355	.....	0.0092	0.0442	0.0534	0.0267	0.0213	0.0480	(b) 336	2.000
29	12	0.074	.....	0.0308	0.0874	0.1182	0.0591	0.0044	0.0635		2.645

(a) Omitted from further consideration at this time because of recent construction. (b) From records; other units, approximations.

GEORGE W. TILLSON, M. AM. SOC. C. E.—In these days of increase in the values of labor and materials, and the prospect that they will continue to increase, it certainly is incumbent on those who utilize these two elements to know that it is done to the best advantage, to see that when they are combined and produce a result, that the result has not cost more than it should. It is not enough for a contractor to know whether a job is a paying one, or a losing one; he should know whether or not every part of his contract is being carried out on a paying basis. Mr. Tillson.

It seems to the speaker that, as Mr. Lewis has said, a great deal of the work which has been done on cost records and reports is more in the nature of accounting than of cost-keeping. Construction work is not done for the purpose of allowing some one to get up an elaborate system of accounting, strange as it may seem to some accountants. The cost-keeping, or the accounting, should be carried on in such a way as to give detailed information about the work, and the important point is to analyze the situation properly into its different elements. By an analysis a contractor can determine very quickly whether a contract is or is not a paying one, if properly carried out. After he does that, he can establish a system of costs for the different movements in carrying out the work. In keeping account of the cost of items, it is possible to go too far; it is possible to build such an elaborate system of cost-keeping that no general result is obtained.

In the Bureau of Highways, in the Borough of Brooklyn, a very good system has been in use for several years, by which it is possible to show the cost of every item in connection with the work in that Bureau. The following from the report of the Bureau for 1911 shows the system of cost-keeping used for the repairs of asphalt pavements, and how the same has been analyzed and subdivided into different items:

ANALYSIS OF MAINTENANCE COSTS.

General and Special Costs.	Wearing Surface. Per Box. (9.3 cu. ft.)
Superintendence	$0.056
Superintendent's auto	0.051
Repairs to plant	0.128
Repairs to tools	0.108
Dumping privilege	0.094
Coal	0.092
Wood	0.046
Misc. supplies, etc.	0.052
Depreciation	0.095
Interest	0.047
Rent	0.033
Carried forward	$0.802

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Material Costs.		Wearing Surface. Per Box. (9.3 cu. ft.)
Brought forward		$0.802
Asphalt	$1.220	
Residuum oil	0.033	
Stone dust	0.195	
Asphalt sand	0.208	
		1.656
Labor and Trucking Costs.		
Plant labor	$0.423	
Street labor	1.694	
Trucking	0.556	
		2.673
Total		$5.131
Cost per cubic foot		$0.552

The total for the 9.3 cu. ft. of material is $\$5.13\frac{1}{10}$ or $55\frac{2}{10}$ cents per cu. ft. It takes about 1.75 cu. ft. for 1 sq. yd. of asphalt 2 in. thick. If the contractor has his work divided on some such principle as that, he can tell absolutely what each item has cost. By daily reports as to the quantity of work done by his different gangs in different parts of the city, he is able every morning to tell whether each man is coming up to his idea of what he should do, or what the records of previous years had shown that men and teams could do.

Mr. Connell.

William H. Connell, Assoc. M. Am. Soc. C. E.—If engineers wish to keep abreast of the times and attain the highest point of efficiency in their work, cost records and reports are essential.

The moral effect alone on an organization is justification for keeping unit cost records. This is illustrated by the pride aroused in the men in connection with their work. The natural instinct of man is to excel in his undertakings, but it must be remembered that men must be led and not driven, and in order to get the best out of them their interest in their work must be aroused, and they must be impressed with a sense of their responsibility. There is no better way of doing this than by the friendly competition resulting from unit cost records. Men are naturally ambitious—this is clearly illustrated in pastimes and sports, whether it be shooting marbles, pitching quoits, playing baseball, or any other form of amusement that brings out competition—all want to excel, and will work hard to do so; one reason is because they understand what they are doing and are interested; the other is because their ambition is aroused through competition, with the result that they will do the best they can. This same situation should exist in all classes of work. There is no man—no matter how lowly he may be, or whatever may be the nature of his work—whose interest cannot be aroused by impressing him with

a sense of his responsibility and showing him wherein the competition lies in connection with his work. Men are often lackadaisical about their work because they do not understand just what part they are playing, and, consequently, have no particular interest in it, whereas if they understood more about the relationship of their work to the undertaking as a whole, their interest would be aroused and they would appreciate their responsibility. The only difference between amusements and work, in bringing out competition, is that in the first instance the man understands the game, has a part in it, and plays it for all it is worth; while in the latter instance he does not understand the work or his part in it; consequently, his interest is not aroused and he does not put forth his greatest efforts, which is not only a loss to him, but to his employers. Therefore, anything that can be done to arouse the interest of the men in their work and bring about friendly competition is of inestimable value, and nothing will do more to produce this result than unit cost records. Mr. Connell.

Cost records are probably the principal means of control of work, and unit cost records are an analysis of cost records, and bring out the weak and strong spots in a working organization. As already stated, unit cost records are simply a modern system of records designed to keep the work and organization up to the highest point of efficiency. Such records are also of great value in comparing costs of similar work in different industrial establishments, municipalities, public works departments, etc. It has often been said that unit cost records do not mean anything on account of the different conditions under which work is done, the difference in wages, labor hours, etc. Of course they are of no value, and are only misleading, unless they are given in detail, but if this is done, and if they are properly compiled, such records are comparable with cost records of similar work in other establishments and localities, and are invaluable. Only a few months ago, the budget for the Highway Bureau of Philadelphia, Pa., for 1913, was made up largely by proportioning the unit cost of similar work in the Borough of The Bronx, New York City. This was made necessary by the absolute lack of cost records or reliable data of any description in connection with the work coming under the jurisdiction of the Bureau. It is impossible to make up an intelligent estimate without the aid of cost records, and it is equally impossible to determine whether all branches of the work are being carried on in an economical manner in the absence of unit cost records.

There seems to be a rather general impression that the necessary job orders, time sheets, material reports, and progress reports which the foreman is required to handle, are apt to confuse him and, consequently, decrease rather than increase the efficiency of his work. This is not so; on the contrary, they tend to develop the foreman, awaken

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his interest, and, in many instances, make a new man of him. Why? Simply because he is placed on his mettle, taken in, to a certain extent, on the financial and business end, made a more important factor in the organization, and, consequently, his work means more to him than the mere physical results. We underestimate the ability of the working classes. A large majority of them, of course, have not had the advantages of an education, but they are all capable of being developed, and are simply waiting for opportunities. Just because a foreman never made up modern cost record reports is no reason why he cannot do it; naturally, it would be confusing in the beginning, and he would make a great many mistakes, but before long, in all probability, he would be suggesting improvements in the form of the reports, which would be better suited to the conditions.

Cost records and reports are invaluable guides in the conduct of work of whatever nature, and must not be underestimated if engineers wish to carry on their work in an efficient and economical manner.

Mr. Blanchard.

A. H. BLANCHARD, M. AM. SOC. C. E.—Labor is an element of road costs which demands careful consideration. Many instances may be cited where equally competent engineering supervision governed work of the same character and where the costs of materials were the same, but where the quality of labor and the number of working hours were not equivalent, and hence the total prices varied to a remarkable extent. The speaker has been connected with work where the hours were the same but, due to incompetency on one hand and ability on the other, the unit labor cost in the former was three times that in the latter case. It is evident that complete data relative to labor should be given, otherwise erroneous deductions will be made.

In connection with construction and maintenance work accomplished under the day-labor system in State, county, and municipal departments, many labor costs are given. Here, without doubt, one finds the greatest variation in the quality of the labor. Poor administration, and political interference with the personnel of the labor organization, are mainly accountable for the marked difference in quality which exists.

Mr. Durham.

H. W. DURHAM, M. AM. SOC. C. E.—One essential feature in regard to cost records is the importance of not letting the keeping of such records become the end instead of the means; undue emphasis should not be placed on the machinery of cost-keeping. Frequently, the matter of recording becomes so laborious, the organization doing it so great, that it is really regarded as the ultimate end to be sought, and, by the time the tabulations are made and the costs worked up, the information obtained is of no particular value on the immediate work.

For efficiency, cost records must be kept so that the results can be compiled at short intervals of time, and give some comparison as to

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the items of the work immediately under consideration, in order that it can be seen instantly whether or not the work is going on properly. It should also be borne in mind that detailed cost records may not be of such great value to the engineer supervising contract work as they would be were he directly handling by day's labor an organization and could point out to the individual gangs, where one excelled another.

For a city or municipal organization, there are no better general methods than those which have been adopted by the Board of Water Supply of New York City, the Public Service Commission, and similar organizations.

Details as to how the records should be kept, what charges should be made, overhead plant charges, etc., are matters which concern all engineers. It is not only necessary to know the price of the materials that are being actually used, but what is being paid for labor and other items, the charge for depreciation on plant, overhead charges, and matters of like character.

Unless such correct distribution of the details of cost is made, the records become worse than useless and allow wrong inferences to be drawn, as in a case of certain road work with which the speaker is familiar, where an excessive unit cost per mile, when analyzed, was shown to be largely due to the inclusion of the entire first cost of the plant used.

Mr. Wilcock.

FREDERICK WILCOCK, ASSOC. M. AM. SOC. C. E.—Determination of engineering costs is important in the public service. Such costs, if compared periodically with the contractors' requisitions for payment for work done, and published for the engineers, are an inducement to eliminate the inefficient members of the corps as soon as possible, and tend to improve the standards of the younger members of the Profession. The elimination is made usually at the end of the probationary period customarily required by civil service rules. This opinion is drawn from observation of construction work in New York City.

(3) DESIGN OF HIGHWAY SYSTEMS.

BY MESSRS. JEAN DE PULLIGNY, BERTRAM BREWER, NELSON P. LEWIS, JAMES OWEN, AMOS SCHAEFFER, J. W. HOWARD, C. E. CARTER, F. O. WHITNEY, AND A. H. BLANCHARD.

Mr. Pulligny. JEAN DE PULLIGNY, M. AM. SOC. C. E.* (by letter).—The writer will not undertake to decide what a highway system ought to be, but will outline briefly what the French highway system is, in the belief that such a system, with appropriate modifications, might furnish a basis for an American centralized State system of highways.

THE FRENCH ROAD SYSTEM.

The following remarks do not apply primarily to the National Main Highways which connect Paris with the large cities and the frontiers, and are constructed and maintained by the Central Government. These main highways (*Routes Nationales*), were built more than 150 years ago—when scarcely any roads were to be found in other countries—for carrying the Royal mail.

Their total length is about 24 000 miles, and the annual appropriation for their reconstruction and maintenance is $6 500 000. The total width of these main highways is 43 ft. 4 in., 53 ft. 4 in., or 81 ft. 3 in., according to classes, including lateral ditches of 6 ft. 8 in. and a central metaled roadway of 16 ft. 8 in., 20 ft., or 25 ft. 6 in., generally constructed of water-bound macadam which is tarred or oiled in summer. On the sides of most of these roads trees are planted, and form graceful avenues, of which many are old and quite beautiful.

The cost of building and maintaining these roads is rather high. It varies, of course, in different parts of France, and, in the past, it has varied according to the times. The general expense for the 24 000 miles of main roads has been about $300 000 000, or an average of $12 600 per mile. The cost of maintenance is about $6 500 000 yearly, averaging $270 per mile. If the average highway of this kind is considered as being about 20 yd. wide, including ditches, the prices per square yard would be $0.35 for construction and $0.00765 for maintenance. If the width is taken at only 15½ yd., the ditches excluded, the maintenance cost would be about $0.0099 per sq. yd.

These highways played a very important part in National activity in France before railroads were constructed. Nowadays, it is believed by the writer, they are less important than the other roads of the

* Most of the information in this discussion has been collected expressly for American engineers by the noted road expert, M. Le Gavrian, of Versailles, *Ingenieur des Ponts et Chaussées*, and Assistant General Secretary of the Permanent International Association of Road Congresses.

country, which amount to 339 500 miles in length and require an annual expenditure of $37 400 000. These roads include mainly *Chemins Vicinaux de Grande Communication,* connecting the cities and villages, and the less important *Chemins Vicinaux Ordinaires,* which connect farms with the next village or the nearest city. Mr. Pulligny.

The total widths of the *Chemins de Grande Communication* are 23 ft. 4 in., 26 ft. 8 in., 33 ft. 4 in., or 43 ft. 4 in., according to the class. These widths include lateral ditches of 3 ft. 4 in. or 5 ft., and a central metaled roadway of 13 ft. 4 in. or 16 ft. 8 in. In the case of the simple *Chemins Ordinaires,* the total width is 20 ft. with ditches of 3 ft. 4 in., and a central roadway of only 10 ft. As to the *Chemins Ruraux,* their dimensions may be still less, and sometimes there are no ditches.

On the National highways the maximum grade is 3 in 100, and the minimum radius of curves is 150 ft. On the other roads the maximum grade is 5 in 100, and the minimum radius of curves is 100 ft. It is important not to increase these grades. On a 3 in 100 grade the effort in hauling a load is about twice as much as that required on a good level road, and on a 5 in 100 grade, the effort is nearly tripled. If the length of such an incline is short, the horses may give the needed supplementary effort, and no reduction of the load is necessary; but if the slope is steep for a long distance, such a reduction is unavoidable. As it applies to the total load, and as the dead load cannot be reduced, it is understood that the useful load is reduced proportionally. Motor trucks are affected as much as horses in such cases, and perhaps more.

Before returning to the subject of main highways, the opinion is expressed that some system of Federal or State main highways, built and maintained by Federal or State engineers, may certainly be justified in the United States, where there is no lack of able engineers, centralized authority, or money. The difficulty is to provide the other system of ordinary roads with able engineers, with centralized authority, and with sufficient yearly appropriations. It is this phase of the subject which will be further considered.

Technical Organization of the French Road System.

France is not so strictly centralized as it is believed to be by many engineers. It is divided into 86 territorial units, called *Departements,* the average area of each of which would be about equal to 3 counties of the State of New York. Hence, New York, having 61 counties, would make 20 French *Departements.* The 86 *Departements* (plus Belfort Territory) are divided into 275 *Arrondissements* (average by *Departement,* $\frac{275}{86} = 3.2$). The 275 *Arrondissements* are divided into 2 325 *Cantons*

Mr. Pulligny. (average by *Departement*, $\frac{2\,325}{86} = 27$; average by *Arrondissement*, $\frac{2\,325}{275} = 8.4$); and the 2 325 *Cantons* are divided into 36 222 *Communes* (average by *Departement*, $\frac{36\,222}{86} = 420$). The gross area of France is 207 000 sq. miles. The average area of an *Arrondissement*, therefore, is $\frac{207\,000}{275} = 750$ sq. miles, being about the average area of a New York county. The area of New York State is 49 204 sq. miles, and the average area of each county is $\frac{49\,204}{61} = 800$ sq. miles.

Each Department is also a unit for several public services, and, furthermore, is a political unit. It has a Governor, appointed by the Central Government, called a *Prefet*, and an elective body called the *Conseil Général*. It has also certain revenues produced by taxes, the appropriation of which is decided by the *Conseil Général*.

All the road system of *Chemins Vicinaux* is managed by the *Prefet*, and the expenditure is voted by the *Conseil Général*, the Central Government having practically nothing to do with it.

The *Prefet*, of course, does not manage the road system himself, but through a centralized body of competent technical men. In about half the Departments the work has been entrusted by the *Conseils Généraux* to the body of Government Engineers—*Ingenieurs des Ponts et Chaussées*—to which the writer has the honor to belong. These roads comprise only a small part of their work. They also have in charge the National Main Highways and the various civil engineering works which are administered by the French Government, including all the inland navigation works, canals and canalized rivers, all the ports, docks, harbors, sea shores, and light-houses, and the close inspection maintained by the French Government over the railroads, with reference to safety, regularity, and rates, and also to secure a proper maintenance of the railroad property which is only entrusted to the railroad companies for a definite period, at the expiration of which, such property will be returned to the Government.

In the other half of the Departments (exactly forty-six) special technical bodies have been organized, which are, of course, quite outside of politics. They include a chief road engineer, residing at the Capital of the Department, near the *Prefet*, and having charge of all the *Chemins Vicinaux* of the *Departement*.

Each *Departement* is divided into three or four political districts headed by a *Sous Prefet*, and called an *Arrondissement*. In each Capital of each district there resides a district road engineer who

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is under the orders of the chief road engineer and has charge of all the *Chemins Vicinaux* of the *Arrondissement.*

Each *Arrondissement* is divided into eight or nine judicial districts, named *Cantons,* each of which also has its small Capital, in which resides an assistant road engineer who has charge of all *Chemins Vicinaux* included in the *Canton.* He is under the orders of the district road engineer.

Finally, all roads in a *Canton* are divided into sections, each having an average length of 4 miles, and on each of these sections the celebrated French *Cantonnier,* or road patrolman, works constantly with his pick-axe, shovel, shrub, and wheel-barrow. These *Cantonniers* are under the orders of the assistant road engineer. A few of them have shorter sections, and they look after the work of their neighbors, as foremen (*Chefs Cantonniers*). The Cantonniers are simple laborers, generally of agricultural training, and are not required to have any special knowledge in order to enter the service. They are only expected to be of respectable behavior, to be able to read and write, and to be steady and trustworthy workers.

It is evident that every square yard of French roads is under the permanent care of a patrolman, of a chief patrolman, of an assistant road engineer, of a district road engineer, and of a chief road engineer. All these men form a hierarchy, with the *Prefet* as the head. Any complaint by the people, or their representatives, to the *Prefet* is properly attended to.

All members of the road service, from the patrolman to the chief engineer, work under a civil service law. When they have once entered the service, they can only be dismissed in case of serious misbehavior. They are promoted at regular intervals, with better pay, and when they retire, after thirty years' work, they get an old age pension.

Most of the patrolmen lack sufficient knowledge to become assistant engineers. The latter are recruited by public competitive examinations, taking place every two or three years, from among young men who have studied, by themselves or in school, the necessary subjects such as the elements of mathematics, surveying, drafting, designing, and road construction and maintenance. The boys employed as helpers for drafting, designing, and surveying, in the offices of road engineers, also generally undergo these public examinations. Their practical experience serves them well, and most of them succeed. The district engineers are generally chosen from among the most able and experienced of the assistant engineers who have had many years of service. The chief engineer for the whole Department may have been previously a district engineer, but it is not obligatory. In some cases he was formerly a civil engineer, a graduate from one of the principal schools, an architect, or an *Ingenieur des Ponts et Chaussées.*

Mr. Pulligny.

Table 1 shows the number of chief, district, and assistant road engineers in each of the 46 Departments where a special road service has been organized.

TABLE 1.—NUMBER OF ENGINEERS, ETC., ON FRENCH ROADS IN DEPARTMENTS HAVING A SPECIAL ROAD SERVICE.

Departments.	Chief Road Engineers.	District Road Engineers.	Assistant Road Engineers.	Field and Office Graduate Assistants.
Ain	1	5	41	46
Aisne	1	6	55	7
Hautes Alpes	1	4	22	8
Ardennes	1	5	32	6
Ariège	1	4	22	11
Aude	1	4	49	16
Calvados	1	6	34	26
Cher	1	2	21	9
Corrèze	1	2	33	7
Creuse	1	4	27	3
Dordogne	1	5	42	21
Doubs	1	5	26	11
Eure	1	7	44	12
Gard	1	4	36	10
Haute Garonne	1	5	48	16
Gironde	1	10	48	48
Hérault	1	5	49	15
Ille-et-Vilaine	1	7	37	16
Indre-et-Loire	1	3	25	15
Isère	1	7	52	30
Jura	1	5	36	10
Landes	1	4	25	9
Lozère	1	2	19	12
Manche	1	6	51	14
Haute Marne	1	4	33	3
Meuse	1	4	39	14
Morbihan	1	6	25	10
Nièvre	1	3	30	9
Nord	1	8	53	28
Orne	1	5	36	15
Puy-de-Dôme	1	5	53	..
Pyrénées-Orientales	1	2	25	6
Haut Rhin	1	..	5	3
Rhône	1	3	32	29
Haute Saône	1	3	34	4
Sarthe	1	5	34	13
Seine-et-Oise	1	7	51	17
Seine-Inférieure	1	5	50	29
Deux Sèvres	1	4	31	12
Somme	1	6	45	15
Tarn	1	5	28	18
Tarn-et-Garonne	1	2	20	13
Vendée	1	8	26	6
Vienne	1	6	32	15
Vosges	1	6	34	6
Yonne	1	6	42	6

As examples, statements will be given concerning the salaries of the road engineers and assistants in two Departments: In Seine-et-Marne (population = 358 325; area = 2 300 sq. miles) the service is entrusted to the *Ingenieurs des Ponts et Chaussées;* in Seine-et-Oise (population = 707 325; area = 2 170 sq. miles) there is a special service of departmental road engineers.

Mr. Pulligny.

Department of Seine-et-Marne.

(Road Service Entrusted to the Government Civil Engineers.)

The body of Government Civil Engineers (*Ponts et Chaussées*) was created on February 1st, 1716, and the *École des Ponts et Chaussées,* for the education of such engineers, was founded in 1750.

The classes and salaries in the body of the *Ponts et Chaussées* are as follows:

	Annual salary.	
Inspecteur Général	$2 900 to $3 400	according to seniority.
Ingénieur en Chef	1 900 to 2 300	" " "
Ingénieur Ordinaire	965 to 1 350	" " "

In addition to their Government work, these engineers are allowed to work for departments, cities, chambers of commerce, etc. From this and from other supplements given by the State itself nearly all engineers earn at least $200 yearly. Many earn more, and the supplements of a few equal or exceed their State salary.

The State salaries of their assistants are as follows:

Conducteurs (Assistant Engineers), $425 to $965 per annum, according to seniority.

Commis (Office and Field Graduate Assistants), $280 to $675 per annum, according to seniority.

After 30 years of employment, all engineers and assistants are granted an old age pension of about one-half of the highest salary they have obtained. Besides their State salary and supplements, the engineers and their assistants receive the following fees for their departmental road service:

		Annual fee.
Ponts et Chaussées:	Chief Engineer	$1 150
	District Engineer	575

Conducteurs (Assistant Engineers)	$123 to $192,	according to seniority.
Office Assistants: Head clerks	210 to 385	" " "
Typewriters and field and office assistants	50 to 150	" " "

The patrolmen are special for the Departmental Road Service. Their monthly salary is as follows, according to seniority:

Chief Patrolman	$25 to $27
Patrolman	19 to 21

When these officials are ordered to travel outside the limits of the city in which they reside, they receive traveling expenses.

During 1912 the total salaries, traveling, and sundry expenses for the road officials, including patrolmen, in the Department of Seine-et-Marne, amounted to $28 000.

Mr. Pulligny.

DEPARTMENT OF SEINE-ET-OISE.

(Special Departmental Road Service.)

The salaries and old age pensions after 30 years' service are as follows:

	Annual salary.			Annual pension after 30 years' service.
Agent Voyer en Chef (Chief Road Engineer)			$1 920	$1 440
District Engineer	$960	to	1 100	825
Assistant Engineer	480	to	850	640
Office and Field Graduate Assistants	280	to	575	430
	Monthly salary.			
Chief Patrolman	$22	to	$25	121
Patrolman	16.50	to	19	100

Many employees also receive various supplements for the high cost of living in certain cities, help to large families, extra work, traveling expenses, etc.

The total of these expenses for the office staff in the Department during 1911 was $10 400. The total expense for patrolmen and chief patrolmen, including all sundry expenses, was $72 500.

These two Departments are near Paris, where the cost of living and all salaries are high. In many other Departments, the salaries would be smaller by 10 and, in a few, by 25 per cent.

The total expense for the *Chemins Vicinaux* of all classes during 1910 amounted to $37 500 000, as follows:

Regular maintenance	$26 355 000
General repairs	2 100 000
Building new roads	4 420 000
Land acquisitions	890 000
Sundry expenses	335 000
Salaries and general expenses	3 400 000
Total	$37 500 000

ADMINISTRATIVE AND FINANCIAL ORGANIZATION OF THE FRENCH ROAD SYSTEM.

The engineers of the road service not only build new roads and maintain existing ones, but take an important part in the administrative and financial working of the road system.

The assistant engineers walk nearly all day on the roads of their district, or they may ride on a bicycle, in a carriage, or in an automobile. They constantly meet the elected Mayors of the small towns, and they know all the needs of the people. Knowing also approximately the available resources for the coming year, they prepare, for each

township and for each road of their district, a scheme for maintenance expenses and for the building of new roads. They send their reports to the district engineer who sums them up and makes any changes he deems necessary. Mr. Pulligny.

All the district engineers forward their reports to the chief engineer, who designs a general scheme for the maintenance and building of new roads in the whole Department. Each town or village has its small elective body which is called to deliberate on the road work and on the expense in which it is concerned. A Bill for this scheme is then discussed by the Department's Legislature in its annual session, and may undergo some changes. The appropriation is finally voted, and the works are then carried out with no more intervention on the part of the political representatives, the road engineers acting only under the authority of the *Prefet*. The expense is levied on the town or village as a public tax, even if the people do not approve of it.

The *Chemins Vicinaux*, thus taken care of by the Department, are the *Chemins de Grande Communication* which connect two or more towns or villages. In such cases it is admitted that the maintenance of the roads must not be entrusted to the townships, because one town might do its share of the work and suffer because the other town would not do the same; therefore, the money is provided by the town taxes, but the direction of the work is assured by the Department.

The construction of new *Chemins de Grande Communication* is undertaken by the Department for the same reason, but, in this case, there is an important difference as to the origin of the funds. Instead of providing all the money from municipal revenues, the towns only give a part of it, and the remainder is appropriated by the Department from its share of certain taxes, the amount of which is divided between the Central State, the Department, and the towns. The sharing of the expense between the town and the Department is provided in accord with definite rules, in which both the needs of the township and its resources are considered.

The total revenue produced by certain taxes is supposed to be an index of the comparative wealth of the townships, and the area of their district is considered as a measure of their needs for roads. The revenue being divided by the area, the quotient is considered as an index number by which the townships are classified, and, for a certain index number, they may receive a definite percentage of help from the Department, as high as 85% for a very poor township with a very wide area needing very long roads.

A similar classification is made in the Departments on the same double basis of wealth and area, and an annual appropriation from the Central Government's fund is divided between the Departments as a National aid for the construction of their roads. This appropriation

Mr. Pulligny. amounted to only $2 000 000 in 1910. It has been larger in certain other years.

Such is the technical, administrative, and financial system, and it has worked satisfactorily in France for nearly a century. It only applies to the *Chemins Vicinaux de Grande Communication* which concern two or more towns or villages.

As for the *Chemins Vicinaux Ordinaires, Chemins Ruraux,* and *Rues* (streets) which concern only one town or village, the Mayors are free to build and maintain these roads out of the municipal funds, as they wish. In fact, all the villages and small towns voluntarily entrust their road work to the assistant road engineer, whom they see daily, and he does it for a small fee. If a town is more important, if it has a few municipal works of sewerage, water, gas, or electricity, to be looked after, besides the road work, a special engineer is appointed and takes care of the whole. If a city is still more important, one or more municipal engineering services are organized. The municipal engineering services of the City of Paris are extremely complete, and their organization is most remarkable, from every point of view.

A few words may be devoted to two other divisions made by the laws of the past in reference to the French roads, namely, the *Routes Departementales* and the *Chemins d'Interet Commun,* which are nothing more than types of *Chemins de Grande Communication.*

The difference in names carries a few changes in the rules governing the management of these roads and the corresponding funds. These changes are not directed toward simplicity. For many years the tendency in all Departments has been to have only one class of roads, the *Chemins de Grande Communication.* No more *Chemins d'Interet Commun* are created, and every year some *Routes Departementales* are dropped from the official lists, and are afterward considered as *Chemins de Grande Communication.* The length of the *Routes Departementales* has decreased from 29 500 miles, in 1869, to 8 100 at the present time.

On January 1st, 1911, the 395 729 miles of *Chemins Vicinaux* were distributed as shown in Table 2.

As previously stated, there are also 8 100 miles of *Routes Departementales* and 24 000 miles of *Routes Nationales,* forming a grand total of 428 000 miles of roads of all classes.

The building and maintenance expenses of the *Chemins Vicinaux* have varied according to time and place, but the figures in Table 3 give an idea of what they usually cost. The lengths considered include only the roads accepted or under construction.

For comparison the figures relating to the *Routes Nationales* have been reproduced, and also some for the *Routes Departementales.*

The total length of French roads is nearly 372 000 miles, and their total cost may be considered roughly as more than $1 500 000 000. The

difference between these 372 000 miles and the total of 428 000 previously given, results from the omission from Table 3 of the 56 240 miles which have only been designed. There are also about 155 000 miles of farm roads, with or without ditches, metaled roadway, and maintenance. Mr. Pulligny.

TABLE 2.—CHEMINS VICINAUX.

Condition.	DE GRANDE COMMUNICATION.	D'INTERET COMMUN.	ORDINAIRES.	TOTALS.
	Miles.	Miles.	Miles.	Miles.
Accepted and regularly maintained............	107 000	47 200	178 000	332 200
Under construction......	287	302	6 700	7 289
Only designed...........	970	1 770	58 500	56 240
Total.............	108 257	49 272	238 200	395 729

TABLE 3.—USUAL COST OF FRENCH ROADS.

Classes.	Total length, in miles.	Average Width, in Yards: Ditches included.	Average Width, in Yards: Ditches excluded.	Approximate Cost of: Building: Total expense.	Building: Per mile.	Building: Per square yard, ditches included.	Annual Maintenance: Total expense.	Annual Maintenance: Per mile.	Annual Maintenance: Per square yard, ditches excluded.
Routes Nationales........	23 800	20	15⅛	$300 000 000	$12 600	$0.36	$6 500 000	$270	$0.0099
Routes Departementales.	8 100	14	11	63 000 000	7 750	0.32	1 500 000	185	0,0095
Chemins Vicinaux de Grande Communication.	107 300	10¾	8½	665 000 000	6 200	0.33	16 900 000	157	0.0105
D'Interet Commun.......	47 500	10	7½	178 000 000	3 750	0.21	6 000 000	126	0.0095
Ordinaires...............	184 700	9	6⅛	457 000 000	2 470	0.16	14 500 000	78	0,0068
Totals................	371 700	....	....	$1 663 000 000	....	....	$45 400 000	....	......

The annual maintenance of the 372 000 miles of regular roads requires nearly $45 500 000, the share of the Central Government being $6 500 000 and that of the 86 *Departements* nearly $39 000 000. This shows a contribution of about $1 per head of population.

For comparison, a few rough figures are given in Table 4 in regard to the other means of public transportation in France, long-distance railroads, local railroads, road cars, canalized rivers, and canals.

Mr. Pulligny.

TABLE 4.—LENGTH, COST, ETC., OF FRENCH RAILROADS, TRAMWAYS, CANALIZED RIVERS, AND CANALS.

	Total building expense.	Total length, in miles.	Average Cost: Building per mile.	Annual Maintenance. Total.	Per mile.
Long-distance railroads...	$3 100 000 000	25 000	$124 000	$31 000 000	$1 240
Local railroads...........	370 000 000	5 200	71 000	1 610 000	310
Tramways (road cars)....		5 750			
Canalized rivers..........	81 000 000	4 350	18 600	1 160 000	266
Canals....................	161 000 000	3 100	52 500	960 000	310
Grand totals..........	....	43 400	....	$34 730 000	....

The total length operated, the total freight, in American tons per mile, and the total revenue of railroads, are given in Table 5.

TABLE 5.—LENGTH, TRAVELERS, FREIGHT, REVENUE, ETC., ON FRENCH RAILROADS, CANALS, ROADS, ETC.

Railroads, Canals, etc.	Length operated, in miles.	Number of travelers at one mile, in millions.	Freight, in millions of ton-miles.	Gross operating revenue.	Net Operating Revenue. Total.	Per mile.
Long-distance railroads (1908)........	25 000	9 900	14 100	$330 000 000	$140 000 000	$5 650
Local railroads (1907)................	4 730	235	105	6 800 000	1 500 000	325
Tramways (road cars) (1907).........	3 130	157	30	3 700 000	500 000	159
Canalized rivers......................	4 350	....	1 940	....	....	.....
Canals................................	3 100	....	2 000	....	....	.....
National roads........................	24 100	....	1 880	....	....	.....
Departmental and vicinal roads......	347 600	....	1 880			

Mr. Brewer.

BERTRAM BREWER, M. AM. SOC. C. E.—In Massachusetts there are Boards of Survey which, to a certain extent, may control the development of the roads in any municipality. Every town has the privilege of accepting a State law which provides for such a Board, and, in many cases, the cities, to which this general law does not apply, have secured from the Legislature a special act governing their own particular cases. These laws vary somewhat, but they are all framed with one end in view, namely, the creation of a board which may guide and, as far as is legally possible, control the design of the various municipal highways or systems of highways.

In the City of Waltham these highways are classed as main thoroughfares or interurban roads, local thoroughfares or parkways, and, thirdly, residential streets. It is the duty of the Local Board to record

Mr. Brewer.

plans, either by adopting the petitioner's designs or by making its own, which will secure a system of highways to fit the peculiar needs of the place. Much can be, and has been, done toward the improvement of municipal street systems. It is a fair question, however, as to whether or not great systems of highways, either State or National, can be built up in this way, though local boards are often broader-minded than one would expect, especially if they are intelligently and tactfully advised by a competent engineer.

The speaker wants to make a plea for more consideration, on the part of engineers, for some of those things which cannot be determined altogether by the transit; a consideration of those factors which make for more permanent values, not only in the road itself, but in the abutting property; a consideration of the fact that, as Stevenson puts it, "every year, as a road goes on, more and more people are found to use it and others are raised up to repair and perpetuate it and keep it alive"; and, he might have added, 'to build beside it".

Two factors which contribute to this permanency of values, not only to the road itself, but also to its surroundings, are often ignored, namely, adequate drainage and the development of suitable and attractive building sites. The engineer fails to remember that drainage of the road itself is no more essential than a location which will admit of inexpensive and adequate drainage of the whole territory which it serves. It is time also for him to remember that even the larger systems of highways will be built on sooner or later, and the character of the building sites which they are destined to develop and serve, on account of appropriate and attractive location, is becoming appreciated more and more by the people of the United States. Highways of any sort cannot be designed and engineered in the same way as a railroad system, for itself and its own particular uses alone. Its surroundings are as important as itself, and too often are entirely overlooked.

Mr. Lewis.

Nelson P. Lewis, M. Am. Soc. C. E.—Highways are intended primarily to supply means of communication and transportation, to permit travel from place to place with a minimum of effort and resistance and in the least possible time. The importance of an intelligent and rational system of roads has not been appreciated. Much attention has been devoted to details of construction, and properly, but there has been too little regard for the real purpose of the highway. Much has been said and written about the planning of the streets of towns, but little about the design of the system of rural highways leading to and from towns and cities, and yet the difference is largely one of scale, rather than of principle. The considerations which should control the development of a system of highways in city, county, or state, have been quite generally disregarded, the chief aim usually having been simply to afford access to the abutting plot, estate, or farm.

Mr. Lewis. The first highways in the United States were developed in a very crude fashion. They followed lines of least resistance; the road surface was the natural soil; the grades were necessarily light, owing to the poor surface, and therefore the roads were circuitous. As the communities which these roads connected grew in population and importance, a better road surface was demanded, but little attention was given to the straightening of lines in order to furnish more direct routes. Little regard has been paid to the general appearance and attractiveness of highways between centers of population. In many cases the main connecting roads lead from some secondary or shabby street in one town or city to a street of the same class in another town; the impression which the user of the road gets of both towns is unfavorable, and the highway loses much in attractiveness and dignity. If such a connecting road approached and entered a town along well-designed roads and streets, increasing in importance and dignity until the civic center was reached, travel over them would be a pleasure as well as a mere attempt to get somewhere.

Every city and town has a straggling system of roads running out into the suburbs, which some day will become important streets, if not thoroughfares, but they are frequently allowed to remain straggling roads until the development of the abutting property has advanced so that a widening and straightening will involve an expense which is prohibitive. To advocate the improvement of such roads in anticipation of future growth requires courage, and will result in criticism from some quarters. To allow them to remain until necessity demands their improvement may be easy, but such a course cannot be justified.

Mr. Owen. JAMES OWEN, M. AM. SOC. C. E.—To appreciate the development of any of the systems of highways in the United States, the condition of the country must be considered. There are perhaps three or four methods of development: In the pioneer development, which took place during the Colonial era, when a man built a house distant from his neighbors, he built a road to it. He generally put his house on a hill. The road finally was extended into a main highway, and that is why a great many of the highways of the Eastern States are not properly located and are bad in grade and alignment; but they are fixtures. The developments along them preclude any main deviation from their original location. About 1820 the Government formulated the purpose of taking the Mississippi Valley and staking out the whole country in one section. The rule was to have a north and south road for each mile. The problems of direction and ease of access were ignored, and even the question of grade was not considered.

The great necessity of wagon travel to-day is through roads in every locality where there is a growing population. There is an utter lack of local appreciation of through highways.

AMOS SCHAEFFER, M. AM. SOC. C. E.—France has undoubtedly the best and most highly organized system of highways of any nation in the world. The systematic construction and maintenance of its public roads probably also dates back farther than that of any other country. The method by which the money is raised to construct and maintain the roads under the jurisdiction of the different authorities is indicated by their classification. Mr. Schaeffer.

Nearly every nation exercises more or less control over its highways, depending on their character and use. Where such control is exercised, the roads are usually divided into two or more classes which indicate whether they are main or secondary highways, and also largely under what governmental jurisdiction they are. This classification is very minutely carried out in France, and represents the relative importance of the roads as arteries of traffic. In most other countries there are not more than two or three classes. Germany is perhaps the most important exception to this method of control. It has no control over its highways, except in Alsace-Lorraine.

In the United States, during the past decade, a great many States have appropriated funds and created organizations for the improvement of the public highways. As these improvements are made, roads pass from local to State control, and the speaker predicts that the time is near at hand when the Federal Government will also take an active part in highway construction. It is even now exerting a great influence by encouraging the improvement of roads in various ways, recommending road materials, superintending the construction of experimental roads, and distributing literature on the subject throughout the country. When the Federal Government does take an active part in road construction, the "Design of Highway Systems" will be on broader lines than has yet been attempted.

In these remarks the speaker will confine himself to the location of roads rather than to the design of the cross-section of the roadway, drainage, road materials, methods of construction, and various other matters which form more particularly a part of the details of the design of highways.

The agitation for better roads has an international scope. The problem of co-ordinating and relocating existing roads, locating new ones to form national highways, and connecting them with those which will be built by other countries is one for the Federal Government to solve rather than the individual State.

The first considerations in the design of highway systems are line and grade, but these are influenced by a great many other circumstances. The most direct line between two large cities may be through a very hilly intervening country, and the best grades which could be obtained on a direct line might be prohibitive to heavy traffic. The

Mr. Schaeffer.

maximum gradient to be used, therefore, will have to be determined before the horizontal location. The adoption of a maximum gradient will possibly increase the distance between the objective points very materially, but distance must always be sacrificed to grade. If a main highway is to be constructed between large cities, some distance apart, there may be smaller intervening cities lying to the right or left of the most direct line which might be located on account of the established gradient. It might be more economical to locate such a highway through these intervening cities, even at the expense of distance, than to connect them with the main highway by branch roads. It might also be desirable to follow along the foot of a hill where protection from the sun in summer and from snow in winter would be afforded by trees and shrubbery, instead of going directly across a barren plain. Where draft animals are used, fresh water may be a consideration to be taken into account. It will be seen, therefore, that there are a great many considerations which exert an influence on the location of a main highway.

The influences which the speaker has outlined as affecting the location of a highway, except the question of grades, apply more particularly to new highways. The routes of existing highways are already determined, so that there is less choice of selection.

The State engineer is often confronted by other problems. It is usually his duty to transform a country earth road into a modern first-class highway. In this case, also, the established maximum gradient should not be exceeded, even at the expense of changing the alignment. The other considerations probably have already been complied with. It may be necessary sometimes to straighten out some bad curves or to widen the roadway at curves, or at intersections, to reduce the danger of collision between fast-moving vehicles. When this is done, a judicious treatment of existing conditions is necessary. For example, an old roadway may be well sheltered by trees which protect both the road and the users of it. It would be folly to cut down these trees for the purpose of widening the roadway at an intersection where it might be possible to construct a new additional roadway behind them. This would provide two roadways, one for each direction of traffic, which would be better than one wide roadway and would save the trees which would give protection to both roadways.

Wherever possible, trees and shrubbery should be saved at all points along a public highway. The esthetic surroundings along such a highway have considerable influence on various phases of life. They will do much toward keeping people in the country instead of having them move to the cities. They also have a tendency to distribute people from the cities in search of recreation instead of concentrating them in amusement and pleasure parks.

The width of a roadway is a question which requires careful consideration, and is governed more by the character of the traffic than by any other factor. A long-distance, through highway, on which the traffic is usually of the high-speed variety, requires less width of roadway than a short road between adjoining villages on which there is a mixed traffic. Two automobiles require less width and less distance to pass than two trucks. A road connecting two villages often accommodates such a variety of traffic as automobiles, trucks, carts, pleasure carriages, perambulators, pedestrians, and children running loose. Such a road must obviously be wider than one which accommodates a less dense and more rapid traffic. Mr. Schaeffer.

In addition to the kind of roads already described, there are those of less importance, which do not connect distant points, but connect smaller places with each other and with the main highways. In the United States these roads are destined to remain under the control of the local authorities for some time to come, and with but comparatively little improvement.

The design of highway systems applies to cities as well as to the country, and some of the requirements for a country highway apply also to city streets. Where it is proposed to lay out a system of streets for a new city, the problem is comparatively easy, but cities, as a rule, have not had their birth by design. The problem of the design of a street system is usually the planning of streets in territory adjoining a city already in existence, or the alteration of streets already built. Where the adjoining territory is undeveloped, the problem is also comparatively easy. Probably the most difficult problems in street design are the cases where a number of adjoining towns are annexed to a larger city. A good example of this is the annexation to New York City of the territory now known as the Boroughs of Brooklyn, The Bronx, Queens, and Richmond. Each of these territories comprised a large area of vacant land over which were scattered a number of small villages, the street system of each of which was oriented differently from that of the original city, and from one another. The solution of this problem is undoubtedly to develop into main arteries of traffic the old highways connecting the villages with the city and with one another by widening them sufficiently to accommodate the dense traffic which is sure to develop. As in country roads, it may also be necessary to straighten them, and this may be done with less regard for the surrounding topography than in the case of country roads, because their character will change as the city expands. The width of the roadway is also governed by the demands of traffic, but that of the street to which the city takes title should be governed in addition by the character of the development which will follow. In New York City, this development will probably be apartment houses, and therefore light, air, and access

Mr. Schaeffer. exert a more controlling influence than traffic. This is a factor which is often lost sight of, even at the present time. The result is that in the future, when expensive buildings occupy the streets, they will have to be widened at great expense.

The following summary, therefore, may be made:

The location and design of international and interstate highways should be undertaken by the Federal Government, that of other roads and streets by the State, county, town, or city, having jurisdiction over them.

The controlling influences on the location of highways are such grades as will be economical, both for traffic and construction, intervening cities and towns, topography, and vegetation.

For through city streets, the controlling influences are directions of route, sufficient width to accommodate traffic, and easy gradients; for streets of less importance, the location is governed by topography, and the frontage which will be provided for private development; the width is governed more by the requirements of light, air, and access, than by those of traffic.

Mr. Howard. J. W. HOWARD, ESQ.—The road problem of a nation or state is entirely different from that of a city. The highway problem has unconsciously been largely discussed from the standpoint of pleasure automobiles, but the important point must not be forgotten, namely, that roads are built to transport vegetable and mineral products.

The German highway system in the Kingdom of Prussia is excellent, and especially in the Province of Hanover where it is designed to obtain the maximum wealth of the soil and convey it by the shortest routes to the centers for further shipment by rail to the ultimate consumers in cities, and for export.

The radial system should be designed to reach out as far as possible from a city. Many fine highways may be built, but unless they are linked with little farms and country places, their real economic purpose will fail. Local roads are part of a system. They can be narrower, and can be constructed with less expensive surfaces than main arteries, but they must be just as viable as the main highways; otherwise, the products of the soil are blocked from coming to the great cities. This system of main roads and laterals likewise distributes the products of the central producing factories back to the people who have extracted and supplied the products of the earth: the true wealth producers of a nation.

Mr. Carter. C. E. CARTER, ASSOC. M. AM. SOC. C. E.—A comprehensive system is needed to connect all the towns throughout the State. The town should connect with the State system, thereby making a tentative system through its limits (considering only the town system, regardless of ownership). At present, a man who owns a small piece of

land divides it into house lots and locates the streets without consideration of the continuity of the highways. If the town system is already laid out, then development should be allowed only along these lines, unless any change suggested would better, or, at least, not be detrimental to, the town system as a whole. This can be accomplished by the town refusing to accept streets or allow municipal improvements to be placed in any street not shown in the system or subsequently approved by the proper authorities. Mr. Carter.

F. O. WHITNEY, M. AM. SOC. C. E.—Like most of the large cities of the United States, Boston is made up of several other cities and towns which have been annexed at different times, and each of these cities and towns contained several centers. In 1891 the State of Massachusetts passed an act requiring the City of Boston to appoint a Board of Survey, the duties of which should be to plan, immediately and as expeditiously as possible, a street system for the whole city, and such a Board was appointed. In 1895 its duties were transferred to the Board of Street Commissioners which has charge of the laying out of all streets in the city, and since that time it has constituted the Board of Survey in compliance with the original law. Mr. Whitney.

The law provided that these streets should be planned, and that their directions, widths, and grades should be determined, plotted, and indicated on maps. This work was done very thoroughly, the undeveloped portions of the outlying districts of the whole city being covered.

Particular attention was given to three classes of streets:

First, the connecting thoroughfares between the different centers of population and business. These were made as direct as possible by widening and straightening the old streets. The new widths were determined by considering the probable future traffic, and the grades were made as easy as possible in consideration of that traffic. Those streets were the arteries.

Second, the residential streets, which filled in the undeveloped spaces between the little villages. Their alignment was less direct. The grades were allowed to be steeper where necessary, and the widths, of course, much less than those of the thoroughfares.

Third, a class of roads which provided for pleasure travel, and was largely in connection with the parkways and the park system reservations of both the State and city parkways.

The law required that whenever public convenience demanded that streets should be constructed and laid out as public highways, the plotted lines and grades should be followed, and that private individuals, in the development of their property, should also conform to the lines thus laid down. The penalty imposed on individuals for not following these lines is that the city is prohibited from placing any public works in private streets which are not constructed on these lines and grades.

Mr. Whitney.

The topography of Boston is such, and the settlement of the territory has been so complete in centers, that it has been impossible to make the city over on any rectangular or other systematic development; but the irregular development has been considered by some as ideal from an esthetic standpoint.

Mr. Blanchard.

A. H. Blanchard, M. Am. Soc. C. E.—Highway systems may be considered from the standpoints of the Nation, State, county, township, city, town, and park. In this discussion the problem of design will be limited to a consideration of National, State, and county or township systems. City and town planning present many features peculiar to each, the consideration of which is not necessary in the design of the systems classified above. The design of highway systems includes consideration of the social, industrial, and agricultural development of a country; the inter-relationship between highways, railways, and waterways; the methods of transportation which will probably be used, such as horse-drawn vehicles, commercial motors, motor-busses, and light railways; and the limitations of grades on the different types of roads and pavements for each kind of traffic.

A national system for the United States would necessarily include trunk highways traversing the country from east to west and from north to south, and passing through the great centers of population, and through trunk highways forming connections between the important cities and these main highways.

State highway systems should be developed on the French plan, that is, main trunk highways constructed at the expense of the State, and county highways constructed by State aid under its supervision. Town highways may or may not be constructed with financial aid from the State, but they should receive the advantages accruing from such aid through the medium of its engineering department, as typified by the practice of the State of New York.

With the adoption of this plan of financing the construction of State, county and town highways, it becomes practicable to design a comprehensive system of State trunk highways connecting all important industrial, agricultural, and social centers.

The county system may be built up within the State system on the same principle, but from the local county standpoint. Under this plan, except in congested districts, town highways will generally be feeders to the State and county systems. It is obvious that the different classes are laid out by the unit of government directly affected. It is likewise apparent that if in a lower unit rests the responsibility covering improvement of highways of general interest to a higher political unit, vexatious and costly delays in the completion of the system of trunk highways will probably occur.

(4) EQUIPMENT FOR THE CONSTRUCTION OF BITUMINOUS SURFACES AND BITUMINOUS PAVEMENTS.

BY MESSRS. FRANCIS P. SMITH, H. B. DROWNE, JAMES L. GAYNOR, H. C. POORE, W. S. GODWIN, W. H. KERSHAW, W. H. FULWEILER, J. A. JOHNSTON, ARTHUR H. BLANCHARD, J. W. HOWARD, PHILIP P. SHARPLES, AND PREVOST HUBBARD.

FRANCIS P. SMITH, M. AM. SOC. C. E.—The earliest bituminous pavements laid in America were made with coal-tar, and the first mixing methods were very crude indeed, hand labor being used almost exclusively. By degrees, suitable machinery was devised, and, as the industry grew and sheet asphalt pavements were introduced and became popular, the development of paving plants was very rapid. In large cities, where the area of pavement to be laid was great, the plants were built as permanent fixtures. Where only a limited amount of work was to be done, the investment was too great, and the semi-portable plant was therefore developed. This was designed so that it could be taken down, shipped to some other place, and set up again, without too great expense. Next, the railway plant, built on flat cars, was devised. This was a great improvement, as it could be taken down or set up in a few hours, and was entirely self-contained. A very large number of plants of this type are now in use in the United States. Mr. Smith.

The permanent, semi-portable, and railway types are used chiefly in paving town and city streets, as hauling is expensive, and, in order to operate economically, the plants must be placed near the street or road to be paved. Where large and heavy machinery is involved, its economical transport, of course, is confined to railroads, which are an absolute necessity where such plants are concerned.

The comparatively recent and rapid development of the bituminous concrete type of construction for country highways has resulted in still another plant. Such highways are frequently, perhaps usually, too distant from railroads to make it possible to locate a mixing plant on the line of the railroad and economically haul the hot mixture from it to the work. Even if this could be done, the contracts, owing to local conditions, are frequently let to small contractors who are not able to purchase large mixing plants. Within the last few years, therefore, a number of plants have been designed which, for lack of a better term, will be described as of the concrete-mixer type. Their output is relatively small, and their use is chiefly confined to highway work. They are not as economical or as complete as the larger plants, but when properly operated, there is no doubt that they have their field of usefulness.

Mr. Smith. As the various types of plants have been described elsewhere, it has seemed more important to the speaker to discuss the principles involved in their design and operation than to attempt to describe them in detail.

Regardless of the type of plant used, three distinct operations are involved in the manufacture of bituminous pavements by the mixing method:

1.—Drying and heating the mineral aggregate,
2.—Preparing and heating the asphalt cement or bituminous cementing material,
3.—Mixing the hot mineral aggregate with the hot asphalt cement.

In a properly designed plant, the machinery for conducting each of these operations must be capable of handling sufficient material to insure the required output. This is a very important consideration, with reference not only to the time required to complete the work, but also to the quality of the mixture. Given ample mixing capacity, but insufficient drying and heating capacity, there will always be a strong temptation to rush the material through the dryer and mix it with the bituminous cement before it is dry enough and hot enough to insure the proper coating of the mineral aggregate. With the conditions reversed, the time of mixing each batch will be reduced in order to keep up with the output of the dryer, and this, of course, will result in a defective mixture in which the bituminous material is not evenly distributed throughout the mass and the various particles are not completely coated.

Where the drying and mixing are done in the same drum, as in the modified concrete-mixer type, the temptation first mentioned is always very strong. Furthermore, it is hard to obtain correctly the temperature of broken stone, under the best conditions, and in most plants of this kind it is difficult to get at the stone to test it, even if the machinery is stopped. Frequent stoppages, of course, will be strongly objected to by the contractor, and, besides, thermometers are frequently broken in this kind of work. Add to this the fact that these plants are usually run by men who are somewhat inexperienced (in fact, this is given by some manufacturers as one of the great arguments in favor of this type of plant), and it is not difficult to account for some of the unsatisfactory results obtained.

To understand more clearly just what is involved in the three operations mentioned, they will be considered separately.

Drying and Heating the Mineral Aggregate.—The mineral aggregate usually consists of sand, broken stone, gravel, or mixtures of these, and, depending on the kind of pavement to be laid, requires heating to a temperature between 250 and 350° Fahr.

Notwithstanding the fact that actual paving work is not usually carried on in rainy weather, it is frequently necessary to run very wet sand or stone through the dryer, owing to its exposure to the weather or because freshly dredged sand is being used. Unless ample drying capacity is provided, therefore, the output of the plant when using wet mineral aggregate will be greatly reduced. Many pavements are laid late in the fall or in early winter, and, under these conditions, much greater drying capacity will be required than in warm weather. Where the conditions vary as above described, or late in the season when the difference in temperature between early morning and noon is very great, any attempt at regulating the drying operation by allotting a fixed time for it, which is adhered to without change during the day's operations, is absurd and useless. The temperature of the mineral aggregate must be tested, preferably with a thermometer, at frequent intervals, and the heating period must be changed from time to time as found necessary, and regulated in accordance with the results obtained. Mr. Smith.

In an asphalt plant, over-heating has to be guarded against carefully and continually, yet the heating capacity of the concrete-mixer type of road plants is usually so limited that the tendency is to underheat rather than over-heat the mineral aggregate. Care should be taken, of course, to prevent over-heating in all cases.

When a mixture of sand and stone, or stone of different sizes is being used, care must also be taken that each batch of material is of relatively the same mesh composition. This requires great care in feeding the mineral aggregate to the dryer, and, unless this point is insisted on, the average contractor will totally neglect it, and the resulting pavement or roadway will be hopelessly lacking in uniformity. Generally speaking, a fine mixture requires a greater quantity of bitumen than a coarse one. With the same quantity of bituminous cement to each batch, marked variations in the mesh composition of the mineral aggregate will result in a pavement which is too rich in some spots and too lean in others. In the case of the bitulithic type of pavement, in which the grading of the mineral aggregate is probably carried to the highest point, uniformity of mesh composition is obtained by screening the heated stone into separate bins and recombining the different sizes in definite proportions by weight.

Preparing and Heating the Asphalt Cement or Bituminous Cementing Material.—In sheet-asphalt paving work, the refined asphalt is usually fluxed to the proper consistency with a heavy residuum oil or flux, although in some cases a ready prepared asphalt cement is used. In road work, a bituminous cement of the proper consistency and requiring no fluxing is almost always used. It should be melted in kettles, preferably of sufficient capacity for the day's run, and constructed so that they will not tend to over-heat or unduly harden

Mr. Smith. the material. The kettles should be fairly deep, rather than shallow, as this will minimize the hardening of the melted mass because of the relatively smaller surface exposed for the evaporation of the lighter oils. The best practice is to apply a slow fire until the material in the kettle is thoroughly melted, and then bring it up to the temperature desired, usually about 300° Fahr. Where the kettle capacity is small and it becomes necessary to re-charge it while in use, the tendency is to increase the heat unduly in order to melt the material rapidly to supply the demands of the mixing plant. This is especially true where a relatively hard and solid bitumen is being used. It must always be remembered that the bituminous materials harden when exposed to long-continued or high heat; and, when, for any reason, the material in a melting kettle is kept heated for an unusual length of time, sufficient flux should be added to restore it to its original consistency. This is well understood in the asphalt paving industry, where the consistency of each kettle of asphalt cement is determined by the penetration machine before it is used, and is brought to the proper point from time to time by the addition of flux.

The average road contractor pays no attention to these details, in fact, is usually ignorant concerning them, and the speaker often wonders that his work turns out as well as it does.

Mixing the Hot Mineral Aggregate with Hot Asphalt Cement.— In the sheet-asphalt paving industry, what is frequently termed a twin pug type of mixer is used. In this there are two parallel shafts, revolved by suitable gearing, and attached to them are blades which intermesh with each other and are at different angles on the shaft (somewhat in the same way that propeller blades are placed), so that they will throw the mixture from the ends and sides toward the middle. Some revolving-drum mixers of the concrete type are provided with blades to facilitate the mixing, and some are not. Some are heated from the outside and in others the products of combustion pass directly through them. The time required for mixing a batch depends on its size and the efficiency of the mixer. This must be determined for each plant, and, once determined, should be adhered to rigidly. In no case should the mixing process be commenced until the mineral aggregate and the bituminous cement are at the proper temperature. Assuming that the mixing period is comparatively short, it is unnecessary, and a distinct disadvantage under ordinary conditions, to have the mixing drum heated or the products of combustion pass through it during the process. The bituminous cement, when applied to the mineral aggregate, coats the particles with an extremely thin layer, therefore, at this stage of the operation, it is peculiarly susceptible to hardening by excess of heat. If the mineral aggregate and bituminous cement are heated to the proper temperature in the first place, the bituminous material will remain sufficiently liquid during the mixing process to

coat the stone properly. If either of them is too cold when they are brought together, they should have remained longer in the dryer or melting tank, and, for the reason above stated, no attempt should be made during the mixing process to bring them up to the temperature which they should have reached before they were mixed. Mr. Smith.

A paving mixture made with too hard an asphalt cement will be very difficult to rake or spread, and impossible to compress properly at the temperatures at which normal mixtures are ordinarily handled on the street. Raising the temperature of the mixture only tends to harden it still further. Mixtures containing too hard an asphalt cement will not give proper service, as they will crack in cold weather and grind out under traffic, and do not provide any margin of safety for offsetting the hardening action of time on all bituminous materials.

Of course, it is not necessary to state that the proportions in each batch should be weighed or measured accurately. In asphalt plants of the best type, each ingredient entering into each batch is weighed carefully. Where measuring is resorted to, this is done accurately by striking off the mineral aggregate carefully in special iron measuring boxes, and filling the asphalt bucket until the contents reach the level of a proper measuring gauge. Contrast this with the usual road contractor's method of operations and one will see a vast difference. In the speaker's opinion, it is not too much to say that uniformly good results will never be obtained until the methods used in constructing bituminous roads are closely patterned after those followed in the asphalt paving industry.

It may be interesting and instructive at this point to discuss somewhat in detail just what quantities of material pass through the three different processes previously mentioned for a given yardage of pavement. For this purpose, consider a sheet-asphalt plant having a capacity of 2 000 sq. yd. of 2-in. wearing surface per working day of 10 hours. The average weight of 1 sq. yd. of surface mixture 2 in. thick when compressed will be 200 lb. The total weight of the output, therefore, will be 2 000 $\times$ 200 = 400 000 lb. This mixture will consist approximately of:

Sand	79 per cent.
Dust or filler	10 " "
Bitumen	11 " "
	100 per cent.

The different portions of the plant, therefore, must be capable of handling the following quantities of material:

Dryer	316 000 lb.	= 126.4	cu. yd. of sand,
Melting tanks	44 000 "	= 22	tons of pure bitumen,
Mixer	400 000 "	= 200	tons of surface mixture.

Mr. Smith. The capacity of the mixer in a plant of the size under discussion is usually rated at 9 cu. ft. This means that the batch dumped into it contains 9 cu. ft. of sand plus the other ingredients. The average weight of dry, hot sand is about 95 lb. per cu. ft. In accordance with the formula previously given, and assuming that a pure asphalt cement is being used, each batch would consist of:

Sand	855 lb.
Filler	108 "
Asphalt cement	119 "
	1 082 lb.

It will take 370 batches of this size to turn out the required quantity of surface mixture. In a 10-hour working day, this means 37 batches per hour, or one batch every 1.62 min. Not less than one full minute, with a mixer speed of from 60 to 80 rev. per min., should be allowed for actually mixing each batch of sheet-asphalt surface mixture. This leaves a total of only 37 sec. in which to charge the mixer with the various ingredients and dump the finished mixture into the wagons. With a well-organized gang and a properly working plant of this type, only 20 sec. are necessary, but it can readily be seen that this is one of the features where seconds count. The mixer capacity of a plant is usually calculated very closely, and this makes it more than ever necessary that the capacity of the melting tank and dryer should be ample in order to furnish a full and uninterrupted supply of hot sand and asphalt cement, as it is almost impossible to make up for delays at the mixer.

Having determined the formula for any piece of work, and knowing the time required for mixing each batch, together with the size of the batch, one can calculate a similar schedule of operations for it, and ascertain just what kind and capacity of plant is suitable for the work. After determining these points, a contractor should never be permitted to mix and lay more pavement in a given time than is called for by the schedule. If he does, it is certain that some portion of the work has been slighted. This should be carried still further, if necessary, by determining the maximum number of batches permissible per hour and never permitting him to exceed this in an endeavor to make up by hasty work in an afternoon what he has lost in the morning on account of unfavorable conditions of any kind.

As stated previously, a standard sheet-asphalt pavement, 2 in. thick, when compressed, weighs about 200 lb. per sq. yd. Road surfaces of broken stone or mixtures of broken stone and sand, 2 in. thick, will vary from 175 to 250 lb. per sq. yd. The denser the mixture and the greater the proportion of large stone, the greater will be the weight per square yard.

After the mixture has been made, delivered to the road, and spread, it is necessary to roll it in order to compress it. Either of two types of rollers may be used for this purpose. The tandem type is generally used in laying sheet-asphalt pavements, and the road roller type (in which the wheels do not track) for highway work. Where the mineral aggregate is composed of small-sized particles and requires to be finished to a very smooth surface, as in city pavements, the tandem type of roller will produce better results and a more even surface. Where the particles of mineral aggregate are large, the type of roller generally used in road work is better for the purpose. It gives a greater kneading action to the large particles, and will effect better compression in such a surface than can be obtained with a tandem roller of the same weight per inch of width of tread. In addition to this, the total width of tread in a road roller is less than in a tandem roller of the same weight, and, therefore, gives a greater pressure per square inch. It is somewhat more difficult to finish the work smoothly with the road roller, but, for ordinary road or highway work, this is not as essential as for city streets. Mr. Smith.

Distributors.—The oiling of roadways results in the formation of a bituminous surface, and would therefore come under the present discussion. The chief piece of apparatus used in this kind of work is a distributing wagon for the oil or bituminous material. Where this oil is so heavy that it will not run freely at ordinary temperatures, it is necessary to provide means for heating it, either in the distributing wagon itself or in the tanks from which this wagon is filled. Such wagons are of two general types: gravity distributors, and pressure distributors.

The gravity distributors apply the oil to the road more or less uniformly and in a film of considerable thickness. This film, if of heavy oil, requires a long time to be absorbed by the roadbed, and, in order to make traffic on it at all agreeable, it is necessary to sprinkle gravel or sand over the surface of the oil as soon as it has been applied, otherwise it will be tracked into houses and spattered over vehicles and people riding in them. With a distributor of this type, the absorption of the oil by the roadbed is accomplished entirely by capillary action.

Pressure distributors are of two kinds. In one the oil is forced through small orifices in the distributing pipe by maintaining air pressure in the top of the tank from which the oil is being drawn. Thus the oil is forced downward to a slight extent into the roadway, but there is still a sticky and oily film on the surface much like that produced by the gravity distributor. A large part of the absorption of the oil by the roadway is accomplished by capillary action, and is more or less slow. Roads treated by this method, though less objectionable than when the gravity distributor is used, will still require the application of a coat of screenings or sand on the surface.

Mr. Smith. In the other type of distributor the oil is atomized by the action of the air pressure, in much the same way as in perfume atomizers and oil burners. When properly designed, distributors of this type will apply to the surface of the road, with considerable force, a mist of very minute oil particles which will be driven downward at a high velocity by the air pressure. In this way the oil will be driven to a considerable depth into the roadway, and the treatment may be regulated so that on the surface of the freshly oiled road there will not be any objectionable quantity of oil, thus obviating the necessity of applying a coat of screenings.

Mr. Drowne. H. B. DROWNE, ASSOC. M. AM. SOC. C. E.—During the summer of 1912, the speaker supervised some construction work on the Service Test Road in Philadelphia, on which a portable mixing plant of small type was used. This mixer is known commercially as the Rapid Heated Mixer.

Essentially, the machine consists of a four-wheeled truck, at one end of which is mounted a boiler and engine, and at the other a small cylindrical rotary mixer. Between the boiler and the mixer is a platform on which the loading is done. The mixing drum is surrounded with a hood, furnishing a heating space of about 3 in. between the shell of the latter and the mixer. The heat for this hood is furnished by a coal fire directly under the mixer. Additional heat is obtained from a kerosene torch which may be inserted within the mixing drum. A vertical blade running spirally around the inside of the drum serves to lift the material from the bottom and carry it to the top of the mixer, as the latter revolves. The material then falls to the bottom, and the same operation is repeated. The discharge spout is fixed in the center of the drum at the rear of the machine. The capacity of the mixer is 12 cu. ft., or a batch of about 1 300 lb., including the bituminous cement.

In this instance, a Topeka pavement was being constructed. The aggregate was composed of a mixture of sand and stone complying with the following specification:

Passing	200-mesh sieve,	5 to 11 per cent.
"	40- " "	18 to 30 " "
"	10- " "	25 to 55 " "
"	4- " "	15 to 22 " "
"	2- " "	not more than 10 per cent.

The various materials were taken from the stock piles in wheelbarrows to the loading platform and dumped into the mixer. The kerosene torch was then inserted within the drum, and, as the mixer revolved, the material cascaded through the flame of the torch and was heated to a temperature of about 212° Fahr. When this temperature was reached, the torch was withdrawn, and the asphalt cement,

which was heated to about 350° Fahr., was poured in. A cover was then placed over the opening in the loading end of the mixer. The batch was allowed to mix with the asphalt for from 1 to 2 min., at the end of which time a perfect mix was secured having a temperature of 250° Fahr. Mr. Drowne.

The output of the machine varied, depending on the dryness of the mineral aggregate. On an average, from four to five batches were mixed per hour. When the materials were warmed before being put in the mixer, six batches per hour were mixed for 1 or 2 hours, or as long as the warm material lasted. One batch of material would lay about 5 sq. yd. of pavement, 2 in. thick when rolled. The pavement was constructed on a macadam foundation, and there was some loss, due to the material being compressed into the voids of the surface. If laid on a concrete foundation, one batch would probably have made a somewhat greater yardage. The manufacturers of this machine claim an output of 250 sq. yd. per 10-hour day per machine. It is generally recommended that these machines be used in pairs. It takes from 6 to 12 min. to bring the material up to the proper temperature before adding the asphalt cement. If only one machine is used, and four or five batches per hour are mixed, the time required for each batch is from 12 to 15 min., therefore, considerable time is wasted by the men working around the machine. If the machines are used in pairs, the men can be kept busy practically all the time, and it is the only economical way to use machines of this type. The output of two of them in a 9-hour day would be about 360 sq. yd. In a bituminous concrete in which the aggregate does not form so dense a mixture as the Topeka, but is composed of larger stone particles, the output of the machine is considerably increased.

If the cost of labor is $2 per day, seven laborers being required, and an engineer at $3.50 per day, the labor cost of the operation would be about 4.8 cents per sq. yd., on a basis of 360 sq. yd. per day. The cost of coal, kerosene for the torch, depreciation of the machine, and supervision, would add probably about 4.5 cents, making a total cost of 9.3 cents per sq. yd. It is essential that the stock piles, and the kettle in which the bituminous material is heated, be near the mixing plant, in order to accomplish economical work.

The importance of drying the material must not be lost sight of in using machines of this type, and this part of the operation takes up the greater part of the time. Some means of heating and drying the materials must be used before putting them into the mixer. On the Philadelphia work wood fires were built in pipes placed under the stock piles. This preliminary heating helped considerably, but if the materials were not covered with tarpaulins at night a slight rain would dampen them to such an extent that the mixing would be seriously delayed. With mixers of this type there is not the slightest danger

Mr. Drowne. of overheating the materials, provided the batches are discharged when they reach the proper temperature; but if they are left in the mixer too long they will become overheated.

This machine is also adaptable for mixing cement concrete. When used for this purpose, a swinging trough is fixed under the discharge spout. The mixed concrete falls from the spout into the trough, which has such a pitch that the concrete runs down to the end and falls on the road. As the trough is 10 ft. long and may be swung through an arc of 180°, a width of about 20 ft. of surface can be covered with concrete at one passage of the machine, without wheeling any of the concrete.

The speaker thinks a plant of this type is a good one for small jobs, or for repair work, where small plant cost is desired. These machines cost about $1 200 each.

Mr. Gaynor. JAMES L. GAYNOR, ESQ.—A rapid mixer was used during October and November for mixing 1½-in. stone with an asphalt cement. On an average, 350 yd. per day were mixed. Instead of using 12 cu. ft. to the batch, about 15 cu. ft. were used, the mixer being loaded full. The stone was bone dry. With a battery of two machines, a capacity of 700 cu. yd. per day should be secured. The labor charge for one machine was about 9 cents per cu. yd.

Mr. Poore. H. C. POORE, JUN. AM. SOC. C. E.—During the season of 1912, the speaker was connected with the construction of about 55 miles of bituminous concrete pavement. This work was done by one of the New England States, which, having had the usual experimental sections under test for the past 4 years, decided to use a cold-mixed bituminous concrete for both resurfacing and new work.

Refined coal-tar was used as a binder, this material being shipped in tank cars to the nearest railway station, where it was steam-heated and barreled by the contractor.

During the height of the season, eighteen mechanical mixers were in operation, laying daily a total of 3 000 ft. of 14-ft. road surface, 2 in. thick.

The type of mixer generally used was one manufactured by the Municipal Engineering and Contracting Company. It is similar to the ½-yd. cube batch cement concrete mixer, and has an oil torch heating device for use when bituminous concrete is mixed. The machine is mounted on four wheels for transportation. It consists of a revolving iron box, mounted on its diagonal axis, and driven with direct gearing by the steam engine mounted on the same frame. The mixed material is discharged by tipping the cube, and the mixing chamber is loaded with a sliding skip operated by a small cable hoist.

An air compressor for supplying the oil torch is belted to the engine. Common fuel oil under air pressure forms a blast in the center of the mixing chamber sufficient to dry the mineral aggregate and expedite the

work during cold weather. As the specification called for a cold mix, the blast was not necessary throughout the day's work. Mr. Poore.

The plants were operated in two ways: either as a portable plant moving along the road each day, or as a stationary unit at the stone crusher.

The first method was followed where the work consisted of scarifying the old macadam and adding a new 2-in. bituminous concrete surface. Set-ups were made at the intervals covered in a day, usually about 200 ft. A 12 by 12-ft. dumping board was used under the loading skip, on which to dump the broken stone hauled from the railroad siding or stone crusher. The mixer was placed on planks in the center of the road, being hauled ahead by the steam roller at the end of the day.

Two 15-gal. portable kettles provided sufficient hot bituminous material. The proper quantity for a batch was measured out by hand and carried in buckets from the side of the road to the loading chute. Six iron barrows, well oiled to prevent the coated stone from sticking, were used in carrying the hot mixture to the road, where it was raked out to the required depth.

One roller was able to prepare the old macadam and roll the mixture delivered each day. When the mixer was used as a permanent unit at the stone-crushing plant, the mixer platform was built at the same elevation as the bottom of the stone bins, and from the latter the stone could be discharged directly to the loading skip. To allow the uncoated stone for the foundation course to fall by gravity into wagons, it was necessary to leave an intervening space of 7 or 8 ft. between the stone bins and the mixer platform. In some cases, however, the mixer was placed on the other side of the bins, and then an extra chute was built to feed into the loading skip.

Many of the mixers ran three ½-cu. yd. batches in from 10 to 15 min. A 1½-cu. yd. bottom-dump wagon was placed to receive these three batches, and then hauled to the road, a 2-mile haul often being economical. There was no chilling of the mixture. A larger kettle was used at the stationary plants than on the resurfacing work. The buckets of hot bituminous material were handed from the ground by the kettleman to the man on the raised mixer platform, and he emptied them into the mixer chute.

One engineer, one helper, one loading man, and one kettleman were required for the mixer when in operation. The broken stone was not touched by hand until raked out on the road.

The cost of the plant was:

Mixer complete....................	$1 550
Two kettles........................	200
Small tools and lumber............	100
	$1 850

Mr. Poore. Practically 600 sq. yd. were turned out per day per plant. On country roads, where a cold-mix bituminous concrete is specified, such an equipment may be considered very satisfactory.

Mr. Godwin. W. S. GODWIN, ESQ.—The cardinal point in bituminous paving is unquestionably uniformity. This applies to all the different methods of construction, surfacing, penetration, and mixing.

Irrespective of the method, cost, or kind of equipment, there are fundamental requirements which must not be ignored, if lasting qualities in the paving are expected. These can only be obtained by the proper use of appliances designed and built by those who are thoroughly familiar with the materials and the methods of construction.

In view of the fact that the cost of the product of the equipment is in inverse ratio to the volume of the output, economical production can only be obtained by having an output so large that the overhead cost does not assume undue proportion.

Cold Surface Treatment.—Bituminous materials can be properly and economically applied cold from a tank wagon drawn by motor or horse. It should have, across the rear, a distributor adjustable in width from 6 to 12 ft., thus avoiding the application of narrow strips. The distributor proper should have small openings or spraying nozzles at short intervals for the entire length, arranged so that they will entirely cover the desired width of roadway, and not apply the material in strips.

Unquestionably, the most difficult factor to control is the uniform distribution of the necessary quantity of bituminous material per square yard. This can be accomplished by keeping a uniform air or steam pressure in the tank, or by a pump connected between the tank and the distributor proper. Unsatisfactory results are obtained with either of these methods if the speed of the wagon or the motor varies. This objection, however, can be overcome by using a specially designed pump connected with the running gear of the wagon or truck, or by drawing the tank wagon by a steam roller at a uniform speed. Any of these distributors should be capable of distributing evenly from ½ to 1½ gal. per sq. yd.

Gravity distributors of any kind have the disadvantage of being able to handle only the light gravity oil; the quantity distributed also gradually decreases as the tank empties.

Hot Surfacing and Penetration Methods.—If the pressure distributors are equipped with interior steam coils, and sufficient steam is supplied to keep the bituminous material at a uniform temperature of about 280° Fahr., they are capable of distributing the heavier grades for either the hot surfacing or the penetration methods of construction.

The equipment generally used in the penetration method has been portable or semi-portable melting kettles, having capacities ranging

from 50 to 500 gal. and hand-distributing pots. This method of heating and applying is expensive and the results obtained are invariably crude. The bituminous material is often too cold, and, in some cases, is overheated and damaged. It is quite apparent that when a pouring pot is used, the uniform distribution of the hot material depends entirely on the workmen. Even with the utmost care, non-uniformity shows on the finished roadway. An excess of bituminous material will collect in spots and hold any kind of stone or dirt, even after the completion of the roadway. This causes an uneven surface which is very objectionable. Mr. Godwin.

The arrangement and equipment for penetration work is most satisfactory when the contractor receives the bituminous material at the nearest railroad siding, in 6 000, 8 000, or 12 000-gal. tank cars, equipped with interior steam coils. A 20-h.p. boiler may be attached to the steam coils in the tank car and thus heat the material to the desired temperature. If this arrangement is provided at the railroad siding, the hot material may be run by gravity into the distributing wagons. If this is not practicable, the material may be pumped from the car to the wagons. A horse-drawn distributing wagon may be hauled from the railroad to the work and then may be attached to a steam roller.

Bituminous material received in barrels costs the contractor an additional sum of at least 2 cents per gal. for each barrel and also the freight on the barrels, which is about 15% of the gross weight. Besides, he has two or more melting kettles to operate and a very large bill for fuel, to heat them.

A 20-h.p. boiler and a 600-gal. distributing wagon will cost about $1 000, and, with an average haul from the railroad siding, should cover 800 sq. yd. per hour. Two 400-gal. melting kettles, at $400 each, and a dozen buckets and pouring pots will cost about $850, and will not cover one-half as great a yardage. It is a case of inferior construction at an exorbitant cost, if the proper equipment is not used.

Should the extent of the work not warrant the purchase of such a plant, there should be secured a strong, well-built 500-gal. melting wagon and a hand distributor, having a capacity of at least 30 gal., mounted on wheels, and having a regulating distributor at least 20 in. wide. A distributor of this kind costs $65, and should pour 250 sq. yd. per hour, using 1½ gal. in the initial pouring and ½ gal. in the flush coat. The use of pouring pots should be avoided if possible.

Mixing Methods.—In the manufacture of bituminous mixtures of any kind, uniformity is the principal factor. This applies, not only to proper proportions of each different ingredient used, but to the temperatures at which they are heated and mixed.

The mineral aggregate, composed of stone, sand, and inorganic dust, must be heated uniformly to the proper temperature, and in

Mr. Godwin.

such a way as to prevent segregation of the larger from the smaller particles. This is a serious matter when different proportions of two or more sizes are used. The aggregates are usually heated in a cylinder dryer, with external heat. The heat is then drawn through the interior, the different sizes, in the proper proportions, being fed continually through the hot cylinder. If a batch heater and mixer is used, the proper proportions are measured or weighed before being placed in the mixer.

The kettles must have sufficient capacity to provide the quantity of bituminous material required to coat the mineral aggregate as it is heated by the dryer. The heat should be uniform and not so great that it will "burn," or harden any portion of the material. The melted material should be agitated slowly by compressed air. This is especially necessary where two kinds are used to obtain the proper consistency. The melting kettles should be protected from the rain, as water not only damages the bituminous material, but causes it to foam and run over the sides of the kettle, in some cases causing serious fires.

The quantity of hot material for each mix or batch should be measured or weighed accurately, and not mixed with the mineral aggregate until the stone or sand is of the proper temperature.

The hot mineral aggregate and the hot bituminous material should be mixed quickly and uniformly, with as little loss of heat as possible. The resulting homogeneous mixture should be spread quickly and evenly with strong iron-shank rakes and thoroughly rolled on a solid foundation.

If a portable batch mixer, having an interior flame in the drum, is used to mix the heated mineral aggregate with the bituminous material, the heat must be cut off before the latter is placed in the mixer, otherwise the excessive heat will damage it. Any method of mixing which allows the bituminous mixtures to come in contact with the flame or a heat in excess of 500° Fahr., if only for a short time, will convert the bituminous material from an adhesive and malleable consistency to a hard and brittle one, entirely unsuited for paving.

To receive their ultimate compression, mixtures similar to bituminous macadam and bituminous concrete, having a large percentage of stone, should have their final rolling with a 3-wheel roller weighing at least 10 tons. If a 5-ton tandem roller is used for the initial rolling, a more uniform surface will be obtained on the finished paving. The same method of rolling should be used for sheet asphalt, but both the rollers should be of the tandem type and weigh 2½ and 7 tons, respectively.

These fundamental principles must be adhered to, in order to obtain uniform and lasting paving, irrespective of the method and equipment used.

Mr. Godwin.

The cost of heating and mixing plants depends principally on their capacity and the care and material used in their construction. A small portable batch heater and mixer, similar to a concrete mixer, and capable of heating and mixing about 7½ tons per hour to a temperature of 300° Fahr., costs $1 500. Mixers of this class are only capable of mixing stone which is larger than ¼ in. As the bituminous material is placed in these mixers hot, a 500-gal. melting kettle is required.

For close or dense mixtures, stationary, semi-portable, and railroad plants are used.

Semi-portable plants, comprising the heating drum, mixer, melting tank, etc., cost $7 500, exclusive of any building, and have a capacity of about 75 sq. yd., or 7½ tons, of sheet-asphalt mixture per hour.

The improved railroad plants, which cost about $12 000, are capable of heating and mixing sufficient asphalt and sand to a temperature of 325° Fahr., to lay 175 sq. yd., or 17½ tons, of sheet asphalt mixture per hour.

The modern duplex stationary plant, in which the large dryers, 15-cu. ft. mixers, conveyors, etc., are operated with independent motors, cost about $33 000, including a steel building. These plants have a capacity of 500 sq. yd., or 50 tons, of sheet-asphalt mixture per hour.

As mixtures of stone are laid at a lower temperature and require less bituminous material than sheet asphalt, the capacity of plants increases about 18% when heating and mixing for paving of this class.

In buying a bituminous mixing plant of any kind, the contractor or municipality should receive bids only from companies which have had considerable experience in the manufacture of such machinery. It should be required that the plant be erected and operated under the direct supervision of the builder until it has met the guaranteed requirements. The guaranty should be for a certain number of pounds of properly heated paving mixture at a specified temperature, per day of 10 hours, and not a certain number of square yards. As all dense bituminous mixtures, when compressed to 2 in., weigh very nearly 200 lb. per sq. yd., this portion of the guaranty can easily be changed from square yards to something which is definite and easily ascertained. The contract should also state the maximum quantity of fuel to be consumed in 24 hours, and last, but not least, the date when the finished plant will be completed, erected, tested, and ready to run to the guaranteed capacity.

Mr. Kershaw.

W. H. Kershaw, Assoc. M. Am. Soc. C. E.—There is one class of equipment which has not been given proper consideration, and that is storage plants for road oil.

Equipment is acquired or improved because of the increased efficiency or saving in cost accomplished by its use. If it was thoroughly understood that equipment for the temporary storage of road oils,

Mr. Kershaw. tars, and asphalts would earn a satisfactory return on the investment, interest in this branch of road work would be increased.

It has been the custom in the past, when buying light oil, heavy binding oil, tar, or asphalt, to order one or more tank-cars, and then hold the cars until the material has been used. With the exception of one or two companies, no charge per day for demurrage on cars has been made for the use of this equipment. A charge of $1 per day is collected by the railroads in all cases, but, of course, none of this goes to the owner of the cars. The economic loss resulting from holding tank-cars out of service has been considered to fall on the shipper, but in the final analysis it is apparent that the price of the oil must cover the loss. Just what this loss of service for a single tank-car amounts to can be better understood when the following figures are considered.

At present, the leasing value of a tank-car is about $1.25 per day; that is, the seller either pays that amount in the form of a lease or, if he owns the equipment, can, in turn, lease it to some one else for that figure. This $1.25 per day practically covers the maintenance and depreciation of the car, and does not include an earning on the equipment which its owner is justified in expecting. A study of Table 1, which gives the time consumed in delivering a full carload (8 000 gal.), by the same car, to cities with and without equipment, shows the injustice of allowing the same tank-car charge to apply for all deliveries.

Table 1 gives an actual record of several cars which were in the road-oil service in the East during 1912, but for convenience the cost figures are based on a leasing value of $1 per day. In considering this record, it must be borne in mind that the producer or seller is carrying his tank-car equipment for 12 months in the year, and during the road season (from 6 to 8 months), is using the cars to the limit of their capacity. For the remainder of the year he has nothing for them to do or is turning them into some branch of the service which does not bring him an adequate return for their use.

From this table it will be noted also that a car can be shipped to a town or a city having equipment, and make a round trip in from 9 to 15 days, at an actual cost of from $12 to $25, delivering in that time a full load of 8 000 gal. When the tank-car charge on business of this class is added to the base price, it makes possible a lower quotation than can be given on deliveries made to points without equipment, where the car is gone from 20 to 100 days, in delivering the same quantity of oil. The shippers are ready to make lower quotations to cities equipped with storage plants, than can be made to points where their cars are delayed.

The fact that the installation of this equipment results in a material saving has been recognized by several of the large users of road oil, as witnessed by the following examples.

District of Columbia.—The District of Columbia has erected six 12 000-gal. and one 15 000-gal. road-oil tanks, these tanks being situated at various points throughout the District convenient for distribution. Three of the tanks are at the Property Yard of the District of Columbia, at 12th and N Streets, Northeast, on a siding of the Baltimore and Ohio Railroad. Fig. 1 shows only the 15 000-gal. tank of this particular group. There are two 12 000-gal. tanks directly under the railroad trestle. All these tanks are fitted with steam coils. Between the two 12 000-gal. tanks, and extending out under the end of the railroad ties, there is a mixing box, containing a mixing pug, and there are both water and oil inlet pipes, for the manufacture of emulsified road oil. Mr. Kershaw.

TABLE 1.—RECORD OF TANK-CARS.

Car No.	Consignee: With or without equipment.	Date shipped.	Date returned.	Number of gallons.	Number of days car is held at oil company's terminal.	Number of days required to make the round trip.	Cost of car, based on leasing value of $1 per day.	Earning capacity of car, in gallons per day.
017060	City (with).........	5/ 1/12	5/ 9/12	8 000	0	8	$8	1 000
017060	Town (witho[illegible]......	5/ 9/12	5/27/12	8 000	0	18	18	444
017060	City "	5/29/12	6/20/12	8 000	2	22	22	363
017060	City (with).........	7/11/12	7/22/12	8 000	*21	11	11	727
017060	City "	7/22/12	7/30/12	8 000	0	8	8	1 000
014480	City (witho[illegible])......	4/15/12	5/ 3/12	8 000	0	18	18	444
014480	City "	6/11/12	7/ 6/12	8 000	†37	25	25	320
014480	City (with).........	7/ 6/12	7/16/12	8 000	0	10	10	800
014480	City "	7/16/12	7/24/12	8 000	0	8	8	1 000
014480	City (withou[illegible])......	8/ 3/12	8/16/12	8 000	9	13	13	615
011150	City (without)......	4/11/12	4/27/12	8 000	0	15	15	533
011150	City "	4/27/12	5/31/12	8 000	0	34	34	235
011150	City "	5/31/12	7/ 2/12	8 000	0	31	31	258
011150	State "	7/ 3/12	8/ 8/12	8 000	1	35	35	228
011150	City (with).........	8/12/12	8/26/12	8 000	4	13	13	616
011150	City "	8/31/12	9/ 9/12	8 000	5	9	9	888
014680	City (without)......	5/ 4/12	5/27/12	8 000	0	22	22	363
014680	City "	5/31/12	6/20/12	8 000	4	20	20	400
014680	City (with)	7/10/12	7/20/12	8 000	*20	9	9	888
14680	Town (without......	7/23/12	9/16/12	8 000	3	54	54	148
013600	City (with).........	5/27/12	6/ 4/12	8 000	0	7	7	1 143
013600	City "	6/ 7/12	6/13/12	8 000	3	5	5	1 600
013600	City "	6/13/12	6/20/12	8 000	0	6	6	1 333
013600	State (without).....	7/ 3/12	8/14/12	8 000	13	42	42	190
014600	City (without)......	4/ 9/12	5/ 4/12	8 000	0	25	25	320
014600	Town "	5/ 4/12	6/ 3/12	8 000	0	30	30	267
014600	Town "	6/18/12	7/ 9/12	8 000	*15	21	21	380
014600	City (with).........	7/ 9/12	7/22/12	8 000	0	13	13	615
019260	Town (without).....	6/25/12	7/11/12	8 000	0	16	16	500
019260	State "	7/11/12	10/18/12	8 000	0	100	100	80

*Car made trip with material other than road oil.
†For repairs.

There is a single 12 000-gal. tank at each of the following outlying points: Chevy Chase, Md., Deanwood, Uniontown, and Tacoma Park,

Mr. Kershaw. D. C. Fig. 2, the 12 000-gal. tank at Tacoma Park, shows the type of equipment in the several suburban districts mentioned. All the tanks have heating coils, and the average cost of the 12 000-gal. tanks in place, complete, including coils and erection charges, was $600.

Springfield, Mass.—The storage equipment at Springfield, Mass. (Fig. 3), consists of two 10 000-gal. tanks, carried on concrete footings. These tanks are arranged so that there is a fall of 2 ft. from the bottom of the tank-car, as it stands on the trestle, to the top of the receiving tank, and a fall of 2 ft. from the bottom of the receiving tank to the top of the distributing wagon. Each tank cost $344, f. o. b. Springfield, Mass., and the cost of installation, including piping, was $812, making the total cost $1 500. These are single-compartment tanks, and have no heating coils.

Borough of The Bronx, New York City.—This Borough bought four 6 000-gal. tanks, formerly used as railroad tank-cars, and assembled them on wooden trestles, arranging the piping so that the oil could be run into any tank desired. This was accomplished by placing a continuous main oil line with a shut-off valve on each tank connection. Figs. 4 and 5 show the arrangement of the tanks and piping.

These four tanks cost $950, f. o. b. New York. The construction of the wood trestles and the concrete foundations carrying them, together with the erection of the tanks and the complete piping, cost $750. The cost of painting the tanks and pipes was $70, making the total cost $1 700 for the complete equipment. The difference in elevation between the railroad track and the bottom of the pit makes it possible to fill the storage tanks and, in turn, load the tank-wagons from storage by gravity. These tanks are at 202d Street and Webster Avenue, on a siding of the New York Central Railroad.

Greenwich, Conn.—The Town of Greenwich, Conn., has a two-compartment tank of 12 000-gal. capacity, one compartment being fitted with steam coils. Fig. 6 shows the arrangements of the tank, which is in the side of a railroad fill, thus allowing enough fall to transfer the oil from the tank-car to the storage tank, and from storage to the distributor, by gravity. The cost of this installation was rather high, owing to the number of concrete abutments required to carry the pipe line from the tank-car to the tank and from the tank to the driveway. The cost was as follows:

Tank, f. o. b. shipping point	$327.96
Freight	52.00
Piping	165.00
Concrete abutments	230.00
Painting	15.00
Extras	5.00
Total	$794.96

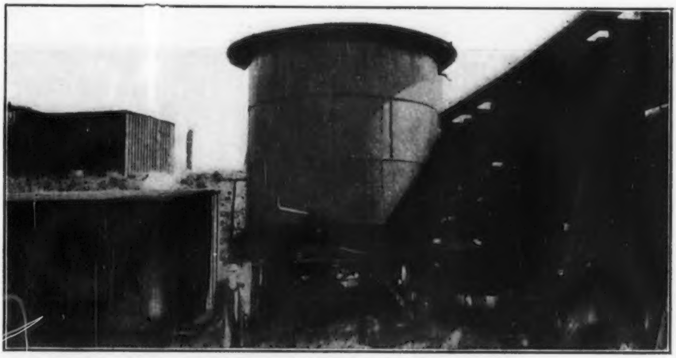


Fig. 1.—15 000-Gal. Storage Tank, 12th and N Streets, Washington, D. C.

Fig. 2.—Road Oil Tank, Tacoma Park, Washington, D. C.

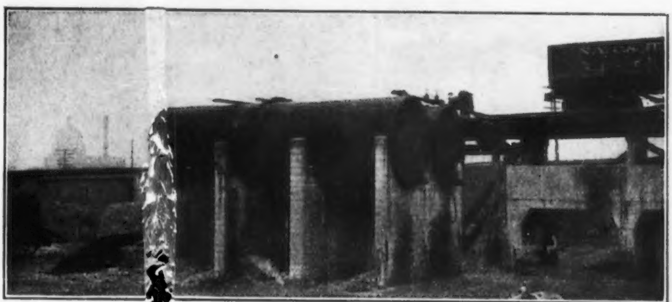


Fig. 3.—Road Oil Storage Tanks, City Yard, Springfield, Mass.

FIG. 4.—ARRANGEMENT OF TANKS.

FIG. 5.—ARRANGEMENT OF PIPING.

MUNICIPAL OIL STORAGE TANKS, BUREAU OF HIGHWAYS, BOROUGH OF THE BRONX.

Fig. 4.—Arrangement of Tanks.

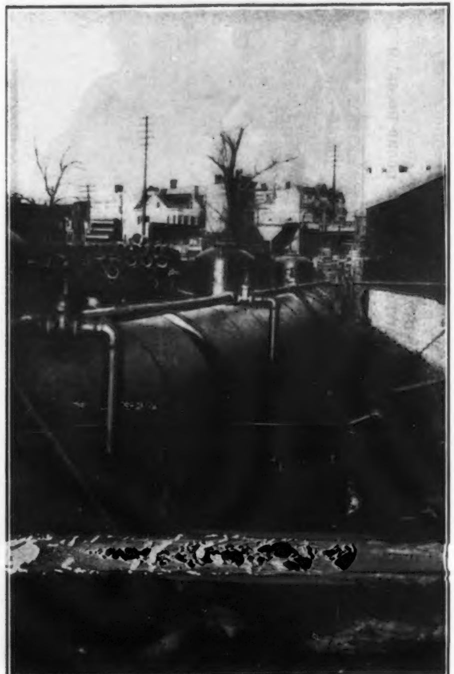


Fig. 5.—Arrangement of Piping.

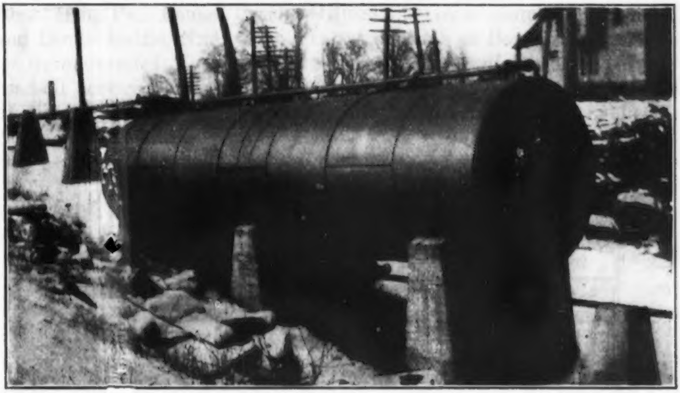


Fig. 6.—Storage Tank for Road Oil, at Greenwich, Conn.

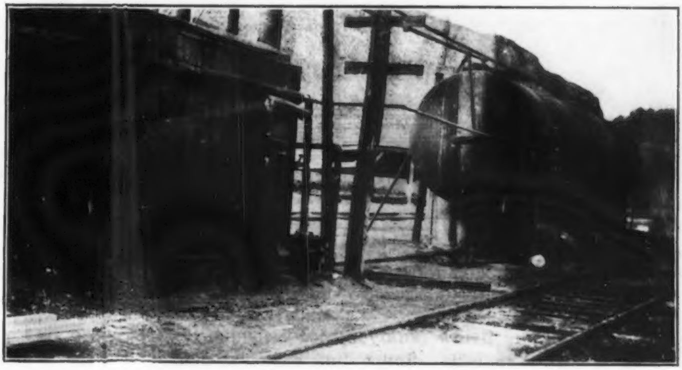
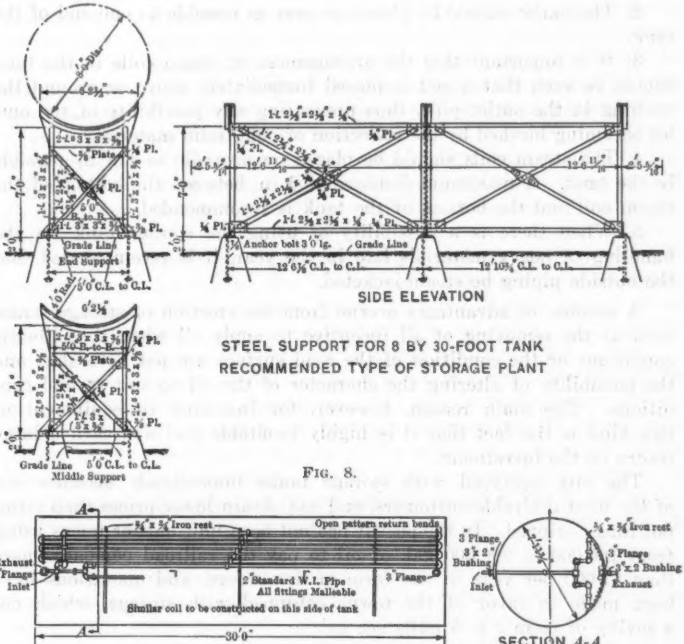


Fig. 7.—Storage Tank for Oil and Asphalt, Erected at Bound Brook, N. J., by the General Crushed Stone Company

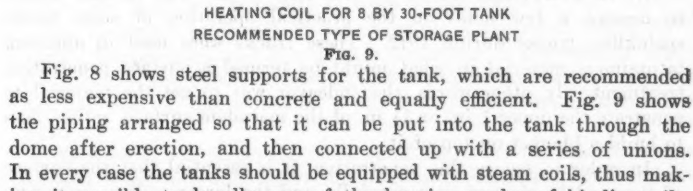
Amies Road Company.—The Amies Road Company has erected seven 12 000-gal. tanks of the type recommended, at Glen Mills and Rock Hill, Pa., Bound Brook, Millington, Great Notch, Springfield, and Lambertsville, N. J. Fig. 7 shows the tank at Bound Brook, N. J. Mr. Kershaw

Recommended Type of Plant.—The capacity of tank-cars in the road-oil service varies from 4 000 to 12 000 gal., hence a tank 8 ft. in diameter and 30 ft. long, with an approximate capacity of 12 000 gal. is recommended.



STEEL SUPPORT FOR 8 BY 30-FOOT TANK
RECOMMENDED TYPE OF STORAGE PLANT

FIG. 8.



HEATING COIL FOR 8 BY 30-FOOT TANK
RECOMMENDED TYPE OF STORAGE PLANT

FIG. 9.

Fig. 8 shows steel supports for the tank, which are recommended as less expensive than concrete and equally efficient. Fig. 9 shows the piping arranged so that it can be put into the tank through the dome after erection, and then connected up with a series of unions. In every case the tanks should be equipped with steam coils, thus making it possible to handle any of the heavier grades of binding oils, tars, or asphalts, as well as light road oils. The cost of these coils

Mr. Kershaw. is nominal, and as these plants are usually constructed in the city yard, where a steam plant is in operation or where a steam roller can be quickly connected, it is a profitable investment to put in the coils when the tanks are erected, even though the possibility of using them for anything but light oil cannot be foreseen. Experience has proven the merit of the following suggestions:

1. The tank should be placed so as to give a minimum fall of 2 in. in 30 ft. toward the outlet end.

2. The outlet should be placed as near as possible to one end of the tank.

3. It is important that the arrangement of steam coils in the tank should be such that a coil is placed immediately above or around the opening in the outlet pipe, thus preventing any possibility of the outlet becoming blocked by the collection of cold, solid material.

4. The steam coils should be placed on a cradle as low as possible in the tank. A maximum distance of $\frac{3}{4}$ in. between the bottom of the steam coil and the bottom of the tank is recommended.

5. When there is a possibility of using the storage tank for the handling of heavy materials late in the year, it is recommended that the outside piping be steam-jacketed.

A number of advantages accrue from the erection of storage tanks, such as the removing of all incentive to apply oil when the climatic conditions or the condition of the road surface are not favorable, and the possibility of altering the character of the oil to suit special conditions. The main reason, however, for investing in equipment of this kind is the fact that it is highly profitable and will earn a large return on the investment.

The city equipped with storage tanks immediately becomes one of the most desirable customers, and can obtain lower prices than cities not thus equipped. In the past it has not been unusual for towns using from 100 000 to 200 000 gal. of oil to pay the railroad company more than $1 000 per year in the form of demurrage, and quotations have been made in favor of the towns equipped with storage, which net a saving of from 2 to 5 mills per gal.

Mr. Fulweiler. W. H. Fulweiler, Assoc. M. Am. Soc. C. E.—The speaker wishes to present a few notes on the practical operation of some motor sprinkling trucks during 1912. These trucks were used in applying bituminous material in what might be termed a surface penetration treatment. In other words, the endeavor was to get the material to penetrate the upper 1 in. or $1\frac{1}{2}$ in. of the macadam surface, rather than to build a blanket on top of it.

In order to secure this penetration, it is essential that the surface of the road be cleaned very carefully and the larger stones composing the wearing course exposed. Even with the most careful sweeping,

it was difficult to secure a surface sufficiently free from dust to allow the uniform application of the bituminous material from gravity wagons, the material being apparently repelled by the microscopic layer of dust on the stone surface. It was evident, therefore, that some form of distributor was required from which the material would be forced under sufficient pressure directly into contact with the stone, so that the dust covering would be brushed to one side. Mr. Fulweiler.

A machine of English type, manufactured by Tar Roads, Limited, London, was purchased. It was provided with a pump geared to the wheels, and by means of the reducing valve enabled a sufficient pressure to be kept on the spray lines, so that with a little care in driving the horses, quite successful results were obtained, but the weight and the small capacity of the tank precluded the use of such an apparatus where there were long hauls from the tank-cars.

With a haul of not more than 1 mile, the horse-drawn type of machine seems to work quite economically, but for a haul of more than 2½ miles, and running up to 15 or 18 miles, it is entirely out of the question.

Two 5-ton chassis, provided with 900-gal. tanks, were equipped with rotary pumps geared to the transmission, through a dog clutch, so that they could be operated when the truck was in motion or at rest. By including a pressure-reducing valve and using a speedometer, it was possible to secure a very uniform rate of distribution.

The quantity of material delivered per square yard depends on the pressure and viscosity of the material, and this, in turn, on the temperature, so that by increasing the pressure or running the truck more slowly in the morning, or during cold weather, it is possible for the driver, after a little practice, to gauge very accurately the quantity he is delivering. The pressure can be varied from 20 to 200 lb. per sq. in., and take care of any ordinary change in the viscosity, due to temperature.

The great difficulty experienced in distributing the material uniformly is mainly in hilly countries. The ordinary motor truck has not enough power to climb the various grades at the same speed, so that it is necessary to change the gear frequently, or climb a hill on a very low gear, which results in considerable loss of time. This requires the driver to do some rapid changing of the relief valves which control the pressure, and considerable experience is necessary to adjust the pressure and speed accurately and economically. It appears that not less than 80 h.p. would be required to climb average hills in the East.

Apparently, the larger the truck (and, therefore, the load), the more efficient it is, but, when operating in country districts, this is limited by the fact that the average township bridge will not carry more than 10 tons. For operation only in the city, or in districts where there is

Mr. Fulweiler.

improved bridge construction, there seems to be no reason why a $6\frac{1}{2}$- or 8-ton truck might not be practicable, but 5 tons seems to be all that can be used in the country.

Another difficulty is due to the wheel base of the truck. On many country roads the shoulders will not bear the concentrated load on the rear wheels, so that it is frequently necessary to run long distances before reaching a suitable point to turn the truck. From 12 to 13 ft. seems to be the wheel base handled most economically on average roads.

In operating trucks to the maximum capacity, the greatest difficulty is due to the fact that the road is not always covered as it should be with a light coating of sand chips or gravel after the application of the material, in order to prevent it from picking up under traffic. In working from one point to another, throughout the country districts, especially about harvest time, it seems to be impossible, or at least very difficult, to secure sufficient labor to apply the covering as fast as the machine will apply the material. In a great majority of cases, the limiting factor for the truck in a day's work is the ability of the laborers to cover the material.

The use of mechanical sand distributors would probably be economical where the haul is short, but where it is from 3 to 4 miles long, it would seem that they are quite out of the question, and that the most economical method is to have the covering material spaced properly along the roadside in piles, about 60 ft. apart, and then hire a gang of from 10 to 12 men and carry them from place to place with the truck to do the spreading. In this way from 2 to 3 miles of 14-ft. road can be treated daily, applying $\frac{1}{3}$ gal. of binder, and from 12 to 18 lb. of covering per sq. yd.

Mr. Johnston.

J. A. JOHNSTON, M. AM. SOC. C. E.—The first pressure-distributing machine brought into Massachusetts by the Highway Commission was not, as has been stated, one in which air pressure was pumped in over the tar, thereby forcing out the tar and oil. The air pressure was not pumped directly into the tank-wagon. The bituminous material was pumped out of the tank, and there was an air chamber attached to the pump to regulate the flow. The Aitken machine (the one referred to) gave fairly good results, but was cumbersome. It held from 1 000 to 1 200 gal., weighed 10 tons when loaded, and was mounted on rather small wheels. Much difficulty was experienced in getting it over unimproved roads in going to and from the State roads, and on many such roads it cut into the soil so deeply that two steam rollers were required to pull it through the bad places.

To overcome these difficulties the speaker devised an apparatus consisting of a steam pump mounted on a separate carriage and fastened behind an ordinary tank-wagon; it was quickly detached from an empty wagon, and re-attached to a full one. The bituminous material was

pumped from the tank-wagon and forced into the road at a pressure of not less than 70 lb. per sq. in. The working principle of the machine is a duplex steam pump with a liberal air chamber; and, to insure practically constant pressure, a release valve is set so that when the pressure exceeds 90 lb., the excess material is forced through a by-pass back into the suction. The quantity of material applied per given area is regulated principally by the rate the wagon travels over the road. There is no speedometer, but the foreman, by the aid of his watch and by counting the revolutions of the wheels, determines how fast the machine should travel. That method seems to be crude, but it has given very good results. It is difficult to calibrate such a machine, because so many factors enter the problem, namely, temperature, viscosity, and kind of material, and such items vary so widely that no set rules can be made, and much must be left to the man on the job. Mr. Johnston.

The method of doing the work is as follows: The bituminous material is shipped to the most available railroad siding in tank-cars. If the material is so heavy that it requires heating before it can be used, the tank-cars are equipped with steam coils, and a portable steam boiler is set up and connected to them for heating. If the bituminous material is tar or oil of about 90% so-called "asphaltic content," the material can be pumped from the top of the tank in about 18 hours after steam has been turned on. A steam pump attached to the boiler is used for this work, and is piped so that the bituminous material is pumped from the top of and returned to the bottom of the car, thus circulating it and overcoming any tendency to stratification. When the bituminous material has attained the proper temperature, it is pumped into tank-wagons holding about 700 gal., which are also equipped with steam coils for re-heating if necessary, and hauled with horses or traction engine to the work. These wagons, which weigh when empty about 3 000 lb., can, when loaded, be easily pulled by a single pair of horses on any ordinary road. The wagons are often jolted over rough surfaces of roads under construction, hence it has been found difficult to keep tight joints in the ordinary steam coil. This trouble has been obviated by using a special form of coil, constructed so that there are no joints within the tank. The coils are fastened securely, and no leaks can occur.

When the wagon arrives at its destination, the horses are unhitched, the wagon is coupled to the steam roller, the sprayer is then attached behind the wagon, steam connections are made to the roller, which then pulls the combination over the road and furnishes steam to drive the pump on the sprayer. The contents of the wagon can be applied on the road in an absolutely uniform coating in 20 min. When the wagon is empty, it is replaced by a full one, and the process continues while the empty tank is returned for refilling. Several tank-wagons

Mr. Johnston. are used, and, with such an equipment, 1 mile of road per day has been treated with a coating of ½ gal. per sq. yd., and covered with pea stone or gravel. Much more could be done if the grit covering could be handled to better advantage. Heating, hauling (3 miles), and applying the bituminous material by this method has been done at a cost of 1¼ cents per gal.

Motor wagons are economical for long hauls from central plants, but, up to a haul of 3 or 4 miles, they cost more to operate than the horse-drawn tank-wagon.

Mr. Blanchard. ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—The selection of plant equipment for the construction and maintenance of bituminous surfaces and bituminous pavements, which will be economically suitable in methods and materials for local requirements in each case, is the key-note of successful work in this field of highway engineering.

A review of the various mixing machines, mechanical distributors, and plant accessories, and a consideration of the many kinds of mineral aggregates and types and grades of bituminous materials used in the construction of bituminous concrete and bituminous macadam pavements, demonstrate that the construction engineer and contractor should have a thorough knowledge of the limitations of each type of machine on the market. To a lesser degree, the same remarks apply to the selection of plant equipment for the construction and maintenance of bituminous surfaces.

If bituminous concrete pavements are divided into three classes, depending on the character of the mineral aggregate, that is, first, those composed of so-called, one-size, crusher-run stone, second, those composed of combinations of stone and sand, and, third, those composed of graded sizes of broken stone or broken stone and sand, it will be seen at once that some machines on the market are adaptable for only one class of pavement, and others may be used for the construction of all classes, although, in certain instances, the overhead charges connected with their operation may not render them economical. Furthermore, when the large variety of solid and semi-solid bituminous cements which may be used for the construction of bituminous pavements, is considered, it is evident that the methods of applying these materials may make it possible to use certain machines and bar others. Such details as the heating of the mineral aggregate may also be a controlling factor in the selection of mixing machines.

In the case of mechanical distributors, the quantity of material required to be applied per square yard and the type and grade of the bituminous material essentially affect the selection. It is well known that, with many distributors on the market, it is impossible to distribute certain materials which may be used satisfactorily for a surface coat, and, with others, it is found impracticable to distribute the small quantities required in some methods of construction and maintenance.

Unfortunately, sufficient investigations have not been conducted by manufacturers of mixing machines or mechanical distributors to enable them to supply all the necessary information to prospective purchasers when definite requirements are submitted to them. As a result, there are many cases of purchases of machinery unsuitable for the work in hand and of unsatisfactory results in construction and maintenance accruing therefrom. Mr. Blanchard.

J. W. Howard, Esq.—The object of spraying is to get the material down into the surface layer before it becomes chilled. That is a mechanical problem. Coal-tars when cool are stiff and brittle, hence with their products it is essential to get them in quickly, while they are liquid and warm. Anything applied by the gravity method is likely to chill and set on top. Almost all asphaltic oils retain fluidity or viscosity, even when cool; therefore, the gravity system often works well with them. Mr. Howard.

High pressures on bituminous products cause the air to enter the products and stiffen them seriously by a so-called oxidation. In the pressure system, care must be taken that the pressure is not so high that it will force air into or through the mass of warmed-up, liquid, bituminous substances, as this permanently stiffens or oxidizes them.

Philip P. Sharples, Esq.*—The speaker believes that better work can be done by some form of pressure distributor than by gravity distribution. It depends, however, on the kind of material used. Some materials work much better with pressure distribution than others on account of being more easily liquefied. With certain kinds of asphalt it would be almost impossible, in the present state of the art, for a contractor to use a pressure distributor. Mr. Sharples.

In Boston a number of Alco trucks have been fitted up with Kinney rotary pumps which deliver the tar to the spreaders at a pressure of about 30 to 35 lb. per sq. in. They, however, are better adapted to seal coats and to surface treatment than to penetration work where a large quantity per yard is called for on the first coat. It has been the practice to use the same pressure on the first penetration coat, that is, putting on about 1½ gal. of tar materials in one coat, but using steam pressure in the tank and a hand hose and spraying nozzle for the distribution. A hose in the hands of a good workman produces very good results. It is absolutely essential to have a good man at the end of the nozzle, or unequal distribution will result. The seal coat may be applied to advantage by pressure spreaders spraying ½ gal. per sq. yd.

The truck is also used on surface coatings, both with hot and cold materials, with excellent results. Auto trucks are well adapted for

* Chief Chemist, Barrett Mfg. Co., Boston, Mass.

Mr. Sharples. central plant distribution, and can cover a territory within a radius of about 60 miles. The material can be kept hot for this distance without any difficulty.

Mr. Hubbard. PREVOST HUBBARD, ASSOC. AM. SOC. C. E.—The thermometer is recognized as an important part of the necessary equipment for plant control. It is rather surprising, however, to note the number of instances where, in the construction of bituminous surfaces and bituminous pavements, the bituminous material is heated in small kettles and no attention paid to the temperature of the tar, oil, or asphalt used. Many contractors and engineers in beginning their work start with thermometers as a part of their equipment, but, as a rule, they are soon broken. The work, however, is often continued without any further attempt to keep temperature records.

The common laboratory thermometer is not satisfactory for obtaining the temperature of large quantities of bituminous material heated in kettles. It is advisable, if not necessary, when obtaining kettle temperatures to use a larger thermometer, preferably one which can be attached to the sides of the kettle and will indicate at all times the temperature of the bituminous material in the kettle. Such thermometers can be purchased at reasonable prices, and have large scales which can be read at a distance, so that the kettle man may see at all times what temperature he is maintaining. For determining the temperature of heated mineral aggregate, the average thermometer is practically useless. If it is continuously thrust into and withdrawn from hot bituminous mixes, it will eventually crack and break, due to sudden cooling.

A thermometer has been designed recently which is preferable to any other that has come to the speaker's notice. It is made by electroplating the lower 6 in. of the tube and bulb of a glass thermometer with a heavy deposit of copper, which protects the glass from abrasion, but allows quick and accurate temperature registration. The thermometer has a sharp pointed bulb, and is made of heavy glass. It reads, from 50° to 500° or 600° Fahr. This thermometer is now manufactured by the Taylor Instrument Company, after a design by Mr. J. O. Hargrove, Inspector of Asphalts and Cements, of the District of Columbia.

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PHYSICAL VALUATION OF RAILROADS.*

By William J. Wilgus, M. Am. Soc. C. E.

With Discussion by Messrs. Maurice G. Parsons, J. Frank Aldrich, J. Shirley Eaton, C. P. Howard, M. H. Brinkley, Albin G. Nicolaysen, F. A. Molitor, Halbert P. Gillette, J. F. Willoughby, S. Whinery, F. Lavis, William W. Crehore, Alexander C. Humphreys, J. H. Gandolfo, T. Kennard Thomson, Charles S. Churchill, R. D. Coombs, Colin M. Ingersoll, Arthur M. Waitt, Stevenson Taylor, F. C. Hand, P. H. Norcross, Charles Rufus Harte, F. W. Green, R. S. McCormick, Charles Corner, and William J. Wilgus.

The Herculean task of valuing the railroads of the United States is of vital importance to the entire nation, whether viewed from the standpoint of rate regulation or its possible outcome, government ownership. The vastness of the interests involved and the effect that the result may have on social conditions, make imperative the adoption at the outset of underlying principles and methods which will appeal to the public and the Courts as being fair and equitable.

It is with the object of promoting a discussion that will be of aid to those burdened with the accomplishment of this task that the writer ventures to advance certain opinions which are the fruit of his experience in the valuation field.

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* Presented at the meeting of October 1st, 1913.

BASIC PRINCIPLES.

There are several ways of determining physical value, depending largely on the purpose for which the result is to be used.

Sale Value.—In taxation matters, the sale value method has been used, right of way and real estate being taken at their assessed valuation, and other items at their depreciated or second-hand value, without the inclusion of overhead and development costs.

In New Jersey this method was adopted in compliance with the general tax law of the State, the expert in charge of the valuation stating in his report as follows:

"Since the real estate and personal property are to be inventoried and appraised separately from the franchise, it seems manifest that it is our duty to consider all of the thousands of parts and items of the property owned by each of the several railroads, entirely apart from their value as considered in the composite whole cemented together by the charter or franchise."

In referring to the fixing of land values, he says:

"We have the right to assume, however, that since the several assessors are elected or selected by the taxpayers of their respective communities, they must have had sufficient experience in land values to entitle them to assess the property in their respective districts, and that, therefore, they are the best available source of information concerning the going price or 'true value' of the land in their districts; and, since the several taxing districts are content to accept the opinions of these assessors as to the 'true value' of the property of individuals, corporations (excepting railroad corporations) and firms, we should give considerable weight to the opinions of the several local assessors when considering the 'true value' of land."

Also:

"No per cent. has been added to the value of lands in excess of the value in exchange for money, as near as such value can be determined by the exchange price of lands adjoining the lands of the railroad in each particular taxing district, and at each particular point where such lands adjoin railroad lands."

While the New Jersey procedure was proper under the conditions there prevailing, it yielded results entirely at variance with those obtained in other States where different theories have guided the appraisals.

Certainly, the sales value principle of treating the component parts of a property as uncorrelated items, at prices that they would bring

if thrown on the market, is not applicable to the ascertainment of the full value of the tangible property of railroads as going concerns.

Original Cost to Date.—In the Act of Congress that prescribes the alternative methods under which the Interstate Commerce Commission is to proceed, Original Cost to Date is named, in which the actual cost of property, as reflected on the books and records of the companies, is to be taken.

With rare exceptions, it is extremely doubtful that the books and records of the railroads of the United States will be found to be dependable for the purpose of ascertaining present-day fair values. Only in recent years have cost accounts been kept in a uniform and complete manner, and even then the almost universal tendency has been to understate charges to construction, and to additions and betterments. This is particularly true on systems that are the product of gradual expansion over terms of many years. Interest during the period of construction, organization and administrative expenses, supervision, freight charges, and the use of equipment on extensions of main line and branches, in many, if not the majority of, instances, have been largely or entirely absorbed in the running expenditures of the parent company, and are not reflected in the book costs. On the older lines the accounts of many of the constituent roads have been lost or are incomplete. Additions and betterments have been charged to operating expenses. Roads have been absorbed through foreclosure proceedings, and the cost has been entered on the books at figures much less than their original cost. In fact, all that can be said of the surviving books of the older railroad systems is that they show merely such portion of the original cost to date as is there recorded, and that, therefore, they are not proper for use in physical valuations, except in so far as they may be of aid in casting a side light on estimated costs of reproduction as of the present time. Moreover, the loss of old records on many roads, and incompleteness of records on others, would necessitate frequent departures from the general rule, with resulting inconsistencies and injustice.

Cost of Reproduction New.—The second of the alternative methods prescribed in the Act of Congress is Cost of Reproduction New. Under this theory of valuation, as generally construed, the configuration of the ground surface and other natural topographical conditions along the line are assumed to be restored and be as they were before the

railroad was constructed, but in all other respects the existing environment is taken as of to-day.

In determining quantities and values under this method it would seem essential that, as far as possible, the procedure should be duplicated that experience has shown would have to be followed, step by step, in the creation of a going railroad. Mere inventorying of the innumerable component items and the placing of a price on each will not give the cost of reproduction, because the intimate relation that the parts bear to each other and the cost of their assemblage into a connected whole are thereby ignored.

The impression prevails that the building of a railroad begins with the breaking of ground and ends with the driving of the last spike. The fact is that there are three well-defined and equally important stages that precede the successful completion of a solvent enterprise, the first and last of which are given but little weight in the usual estimating of cost.

First there is initiation. The plan has inception in the minds of men who are fitted by experience, acquaintanceship, resourcefulness, courage, and tact, to give it life and sustain it through many vicissitudes to final issue. Reconnoissances, preliminary surveys, and estimates of cost, revenue, and profits, are needed to demonstrate its feasibility. Tentative agreements are essential for the features that must be secured before publicity. Arrangements must be concluded for financing the project through underwritings or by sales of securities on a commission basis. Capable men must be selected for the administrative and engineering staffs. All this requires much time, often years, and considerable expenditures which the Interstate Commerce Commission has authorized as chargeable under its classification of "47, Interest and Commissions" and "48, Other Expenditures."

Next comes the constructive stage. This embraces final location, investigation of subsurface conditions, and surveys; the preparation of maps, plans, specifications, and estimates; the purchase and condemnation of rights of way and real estate; negotiations with other railroads for trackage and crossings, and with municipalities and towns for franchises and other privileges; the requesting of bids and the awarding of contracts; and then active construction by contract and company forces.

The acquisition of land and of rights from others is a slow and tedious process, fraught with delays and expensive litigation, and it is rarely safe to proceed with actual construction on important work until this step is concluded.

Construction proper necessarily would not be started simultaneously on all portions of a large system. The main stem would require first attention, followed soon by the important terminals, next the principal branches, then the lesser ones, and lastly the minor branches and spurs.

Expeditious construction of a great railroad system through populous communities does not admit of the awarding of small contracts, as is usual on minor extensions of a "going" railroad. Responsible contractors of wide experience, with followings of sub-contractors, must be attracted by the prospect of work of magnitude at remunerative prices.

Lastly is the educational and development stage, when the line is opened to traffic. Competent forces must be selected, organized, and trained to maintain and operate tracks, structures, rolling stock, and floating equipment. Errors of design and construction, as developed by operation, must be rectified. Equipment in consignments must be received from the manufacturer, "broken in," and assigned to varying duties. Traffic must be induced, economies studied, experiments tried, and the public brought to an appreciation of the new facilities. Stated differently, all parts of the new creation must be co-ordinated and thoroughly trained in their new duties before it is possible to handle successfully a great volume of traffic. This period of education, adjustment, and development, during which the earnings from operation would increase from nothing at the commencement to the full amount at the end of the period, varies, of course, with the nature of the country and traffic. Under the most favorable auspices, it usually lasts for several years.

It is evident that the mere inventorying of the items of a railroad as parts of an inert whole will not give due weight to the processes that experience has shown are essential to the creation *de novo* of a vitalized homogeneous system of transportation, equipped, manned, and trained for the same character of service that it now performs. The problem should be approached in much the same spirit as in the actual organizing, building, and launching of a new railroad.

Cost of Reproduction, Less Depreciation.—The remaining method mentioned in the Act is Cost of Reproduction, Less Depreciation. For rate-making purposes it does not seem to the writer that the physical depreciation should be deducted from the cost of reproduction new. Both law and practice have determined that all expenditures for renewals and repairs should be charged to working expenses, and not to capital; or, stated differently, normal depreciation is not to be treated as a wastage of capital, but as an element in the cost of operation that is covered by the rate.

That any other course would be improper and even illogical is at once apparent when it is realized that in any subsequent reappraisal, say a year hence, the inclusion therein at full value of a renewed item that appears in the appraisal at its depreciated value, practically would amount to the capitalizing of a renewal expenditure that had been charged to income.

In fact, the Act, in requiring that valuations shall be revised from time to time as improvements or other changes are made, in effect prohibits the deduction of depreciation, as otherwise the constant addition of amounts expended in the replacement or renewal of items that had been included in the valuation at their depreciated price, would be a violation of the rule of the Interstate Commerce Commission that operating expenses should not be charged to capital.

For instance, expensive coal pockets which will cost, say, \$600 000 to renew, may be worth a nominal sum in their present-day run-down condition. The adoption of a depreciated figure would mean that when the structure is rebuilt, say, next year, \$600 000 would be added to the valuation, although correct accounting would demand that the entire cost should be charged to operating expenses.

The conclusion seems warranted that, for rate-making purposes, depreciation should not be deducted from the cost of reproduction. It is plainly a liability of the stockholders, partly offset by any reserves that may have been established for that purpose, and should be given due weight in determining a railroad company's net profit and loss account. Of course, in the case of Government acquisition, this feature would require due attention, for the reason that the purchaser would be assuming the stockholders' burden of future restoration of depreciated items.

Summary of Basic Principles.—If the foregoing reasoning is sound, it is evident that the selection of the underlying basic principle must be guided by the purpose for which the valuation is made.

The sale-value method, while used in certain instances where the general taxation laws thus require, is inapplicable for rate-making and acquisition purposes because it does not take into account the features that mark the difference between an aggregation of dissociated parts and the same parts cemented together and pulsing with life as a going concern.

The original cost-to-date method is inadmissible, except as a side light on other methods, because the incompleteness and non-uniformity of past records are a bar to the making of accurate statements of full cost, and because the loss of such records on many roads makes impossible that universality of application that is essential to equity and fairness.

Cost of reproduction new, if estimated in such manner as to embrace all the steps that experience has shown are needed for the creation of a living, self-supporting organism, may be considered as a fair measure of physical value in connection with rate regulation; but physical depreciation, being an element of waste chargeable to operating expenses and therefore a stockholders' liability, is not deductible except in cases where the burden of restoring the depreciation is transferred to a purchaser.

LAND VALUES.

Even though the reproductive principle is accepted, there are many who contend that land values should be entered in the estimate at their original cost, or at the current normal market price of neighboring lands, without any increase for severance and other damages, or for the excess that railroads are compelled to pay for right of way.

Original Cost.—The "original cost" measure of land values is believed to be unfair, for two reasons: It ignores the increment that is enjoyed by all other property owners through increase in the population and prosperity of the country; and it would lead to the inconsistency of low original values on lines unfortunate enough to have preserved their land records, and high costs on adjacent lines where the absence of records would make imperative the use of present-day values.

Current Normal Market Prices.—That current normal market prices are not a proper gauge of the cost of reproducing railroad lands

is well known to all who have had experience in right-of-way matters. From a variety of causes, such as severance and other damages, plottage, and the psychological attitude of owners toward corporate purchasers, the prices paid by railroads are far in excess of normal neighboring values, in cities and towns the average factor ranging from $1\frac{1}{2}$ to 2, and in the country from $2\frac{1}{2}$ to $3\frac{1}{2}$. To ignore this condition is unjust in theory, and in practice would result in confiscation of property where actual expenditures for right of way on recently built lines can be proven to have been many times the cost that would result from the use of fictitious "normal market prices."

Reproductive Prices.—The conclusion seems warranted that strict equity can be served only by the adoption of one standard of land values applicable alike to all roads, and that the fairest standard is reproductive cost, which, as its name implies, calls for the estimated cost of reproducing the lands in each locality at the present time, taking into consideration all the elements which are known to have a bearing on the purchase of railroad right of way and real estate, including costs of acquisition.

This method guarantees equal consideration to the older lines and to those more recently built, avoids the dangers of confiscatory action, and gives due consideration to the facts that are known to enter into the purchasing and condemnation of lands for railroad purposes.

Lands Held in Reserve.—Recognizing that the erection of manufactories and other buildings, the opening of new streets, the laying out of parks, and the making of other costly improvements in the neighborhood of railroads, as well as increases in the values of adjoining property, will make the future acquisition of lands for the expanding needs of common carriers difficult and expensive, if not impossible, it has been customary for railroads to exercise foresight in the purchase of surplus lands at crucial points. It seems that lands in good faith so held in reserve for future use, are necessary for the proper performance of the duties of the railroads as common carriers, and, therefore, should be included in the valuation.

Inventorying and Pricing Measurable Items.

A physical examination of a railroad at once reveals a multitude of items which are patent to the eye and, therefore, may be termed "measurable items."

Interstate Commerce Commission Classification.—To estimate properly the cost of reproducing these items in place, as parts of a connected whole, it is exceedingly necessary that they shall be listed in such a manner that their relation to each other will be apparent; and this can be best done by adhering strictly to the classification of expenditures for road and equipment, as prescribed by the Interstate Commerce Commission. In fact, this procedure is imposed under the terms of the Act.

A number of valuations have generalized their treatment of the data embraced under the primary accounts of the Commission, with results that will not bear analysis. For this reason it is essential that every road item should be entered under its proper sub-primary account, exactly as would be done in the building of a new line. Bunching items together at lump prices will not accomplish this result.

Tentative Prices.—By inventorying all parts, with a view to their relation to each other during the process of construction, and by adhering strictly to the Interstate Commerce Commission classification, it is possible to determine in advance of the field work a list of tentative prices for all the items chargeable to each of the sub-primary accounts that actual experience has shown will be encountered.

As conditions on many railroads in various parts of the country differ widely, each locality requires special study in conjunction with such records of actual cost as may be available. Based on this study, a tentative price list may be adopted for the general information and guidance of the engineer in direct charge of each railroad subdivision. Such prices would be modified by him whenever local conditions made that course necessary, and in such cases his reasons for the modifications should be given for *visé* by the engineer in general charge.

In fixing tentative prices, due consideration should be given to many elements of cost which are of a temporary nature during construction and are not always apparent in after years, as, for instance, salaries and expenses in connection with the acquisition of lands; temporary trestles to facilitate track-laying in advance of the erection of bridge superstructures; preliminary surfacing of track for the preservation of rail from injury pending final ballasting; and transportation of men, tools, and material to points of distribution, the expenses of which are best incorporated in the unit prices rather

than to attempt to segregate them under Interstate Commerce Commission Account 32.

Owing to wide fluctuations in the costs of labor and material, the practice of adopting unit prices that will be fair averages over a period of several years has been followed extensively. This is undoubtedly the proper course, with the exception of items that have shown a constant tendency in one direction, as in the cases of lumber, piling, ties, and many classes of labor. In the latter instances, it would seem that current instead of average prices should be adopted.

Overhead Costs.

(Exclusive of Interest During Construction.)

As already stated, the inventorying and pricing of measurable items by no means yield the full cost of reproducing a railroad.

The necessity of providing means for cementing together these items is recognized by the Interstate Commerce Commission through certain accounts in the classification of expenditures for road and equipment, viz.: 1—Engineering; 35—Earnings and Operating Expenses during Construction; and 43 to 48, inclusive, General Expenditures.

An estimate of the cost of a railroad also requires an allowance for contingencies, which the fallibility of the best of engineering forecasts has shown to be necessary to cover omissions, errors, and uncertainties.

In the aggregate, these overhead expenses constitute a large percentage of the total cost.

Engineering.—It should be borne in mind that Interstate Commerce Commission Account 1 calls for engineering expenses in connection with Road Items 4 to 31 only, and does not apply to Items 2—Right of Way and Station Grounds, and 3—Real Estate, nor to Items 37 to 42, inclusive, Equipment.

Contingencies.—Despite the utmost care in estimating, experience has shown that final costs are invariably far in excess of preliminary forecasts, unless a percentage is added to cover a multitude of contingencies that arise during the prosecution of work.

An honestly prepared estimate never over-states quantities, but, on the contrary, omits many which are overlooked or escape attention.

Claims are made by neighboring land owners, employees, contractors, and others that frequently involve lawsuits.

Condemnation proceedings result in awards for damages far in excess of usual anticipations.

Floods, wash-outs, and errors of employees cause added expense.

Delays are sure to arise that enhance costs, as, for instance, through strikes, court proceedings, and difficulties in securing labor and materials, and also through increased interest charges pending the time of final completion.

Finally, there is always the danger of rising markets for material and labor, and of a scarcity of high-grade contractors.

Of recent years there has been a growing appreciation of the necessity of adding for contingencies not less than from 15 to 20% to carefully prepared preliminary estimates of cost. The instances are many where enterprises have under-estimated their cash needs, through inadequate allowance for this feature, with disastrous financial results.

It has been said that the knowledge gained from the study of a completed railroad largely eliminates elements of uncertainty. This is true of such features as adequacy of waterways, miscellaneous items often forgotten or overlooked in a preliminary estimate, and, to some extent, nature of foundations and classification of excavation; but the largest elements of chance still remain, such as claims, delays, rising markets, scarcity of competent contractors, omissions, inflated land values and damages, complications in connection with the elimination of grade crossings, and the nature of foundations, of which, on the majority of the older roads, little authentic information now exists.

There are also the further items, usually unprovided for elsewhere, of cost of acquiring lands; the expense of organizing maintenance and operating forces pending the commencement of regular operation; and expenditures for engineering, inspection, and "breaking in" of rolling stock.

General Expenditures.—In the words of the Chairman of the Public Service Commission of New York in the Rochester, Corning, Elmira Traction Company decision:

"When the amount of the actual cost of the physical construction of the proposed road has been determined we are still far from having determined the amount of capitalization which should be allowed. There are many elements of cost attendant upon bringing into existence of a new railroad additional to the cost of mere physical construction. Some of these elements may be enumerated as follows: (1) expense of organization, (2) incorporation tax, (3) expense of

obtaining a certificate of public convenience and necessity, (4) preliminary engineering expenses, (5) expense of procuring the authorization of issue of stock and bonds, (6) expense of marketing the securities, (7) discount upon the bonds provided they cannot be sold at par, (8) interest upon the bond issue during the period of construction and prior to the beginning of operations, (9) compensation of officers of the road during the construction period, (10) incidental expenses during construction period, (11) expense of obtaining local franchises and consents.

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"Another subject of great interest and importance is the compensation, if any, to which the promoters of the enterprise should be entitled for their services. Promotion has been so extensively abused and has been so universally used as a cover for abuses in capitalization that it has come to be regarded as a term of reproach and as a device to work schemes of robbery upon the investing public. No reason is apparent why this should necessarily be so. The honest services of a capable promoter are indispensable to the flotation of every comprehensive and far-reaching scheme of development in the railroad world, or elsewhere. A clear vision to see opportunities, ability to demonstrate them to others, and energy to push to completion works untried but of great moment, are indispensable to material development and should be fairly and even liberally rewarded by the public which receives the benefit of those works. Such rewards, however, should be put upon a clear basis of business principle, should be of sufficient magnitude to encourage rather than discourage enterprise, and should not be so great as to make exorbitant demand which is perpetual in its nature, upon the community to be served. They are to be treated simply as just payments for services performed for the corporation, which services are valuable and in many cases even indispensable. Such services should be paid for upon the basis of what they are fairly worth, having regard to all the circumstances of the case

"These observations are so elementary that further elaboration of the principle involved should not be necessary. * * *"

The Interstate Commerce Commission classification of expenditures for road and equipment provides for the charging of general expenditures to the following accounts:

43—Law Expenses;

44—Stationery and Printing;

45—Insurance;

46—Taxes;

47—Interest and Commissions;

48—Other Expenditures.

It has been usual in the valuation of public utilities to fix arbitrarily a percentage of the total cost to cover these items, but it seems possible to make an analysis of these charges and thereby reach a result that may be said to have a logical basis.

Law expenses, stationery and printing, and insurance may be estimated by taking the aggregate annual current expenses for these items on a given railroad and making a suitable allowance for the additional expense that would be probable under the conditions of construction, as compared with normal operation, for one-half the adopted total period of construction from the inception to the final completion of the enterprise.

Taxes during the constructive period may be taken at the current annual amount for one-half of the period extending from the date of commencement of right-of-way purchases to the time of commencement of operation.

Discount on bonds by many is not considered to be properly chargeable to cost of construction, on the theory that pure discount is equivalent to an adjustment of the interest rate to current market conditions. A banker's commission for the sale of securities, however, should be included. Payment for the services of those undertaking the sale of stocks and bonds seems to be just as legitimate and necessary as compensation for other services incident to the carrying out of the project.

"Other expenditures" may be obtained by taking the current annual cost of salaries and expenses of general officers and clerks for one-half of a period extending from the commencement of right-of-way purchases to the commencement of operation. In addition to this there would be the further payment properly due to those who, through their initiative, originality, ability, and assumption of responsibility of final success, would give life to the enterprise. The propriety of making this allowance is well expressed in the above-quoted opinion of the Chairman of the Public Service Commission of the State of New York.

Interest During Construction.

It is generally conceded that interest on money during construction is as much an element in arriving at the cost of the reproduction of a railroad as expenditures for any other purpose in achieving the same result. Payments for the use of money are in principle not different from payments for labor, or materials, or right of way.

During the process of construction of a new enterprise there is no fund on which the company can draw for interest payments, except interest-bearing borrowings supplemented by such net earnings as may result from operation during that time; and hence the capital raised must be sufficiently large to cover all items of cost, including interest upon itself, and "interest on the interest," up to the point of final completion of the road in running order.

Methods of Calculating Interest.—Although the propriety of including interest on money during construction as an element of cost is thus recognized, the tendency has been greatly to under-estimate the amount.

It has been customary to take the estimated cost at the adopted rate of interest for one-half of the assumed period of construction. This treatment is on the theory that the expenditures will increase in a constant ratio or straight line, from nothing at the beginning to the full amount at the end of the constructional period; that money advanced to the company is non-interest-bearing until actually disbursed; and that no interest is paid on the interest payments.

An analysis of the conditions that actually obtain in the building of a large railroad shows that the usual method of calculating interest charges yields a result very much lower than would actually occur in a reproduction of the system, for the following reasons:

1st.—Expenditures do not increase in a constant ratio. In the preliminary stage the rate of expenditures is small. Then there follows a marked rise because of heavy right-of-way payments, followed by a somewhat decreasing rate. Finally, the rate again increases by reason of large payments for equipment. The usual result is a much larger volume of expenditures in the early stages of the work than appears from the use of the straight-line method.

2d.—The securing of capital on a basis that will assure its delivery exactly as needed for disbursement during the progress of the work is not practicable. Stringency in the money market or other unforeseen contingencies make imperative the possession by the company of ample funds to bridge over temporary crises, even when the funds are supplied through banking houses of unquestioned strength. Either the entire amount must be taken at the time of financing, or the capital must be paid in annual installments in advance. In either case the

unused surplus over current needs is loanable at rates materially less than those paid by the company, the effect of which is an addition to the burden of interest charges during construction.

3d.—Interest charges must be paid from the capital, which itself is interest-bearing, and therefore it is not proper that interest on the interest should be omitted from the calculation of this item of cost.

Summarizing, the ordinary method of calculating interest charges usually produces an amount that is too low, by reason of the failure to take into consideration the heavy right-of-way expenditures in the early stages of the work; by reason of the neglect to include interest on unused money awaiting active use; and by reason of the omission of interest on interest.

Rates of Interest.—The rate of interest paid on the money invested has been usually taken at 6%, on the theory that the enterprise would be financed one-half on bonds bearing 5% and one-half on stock entitled to 7%, both bringing par on the market. Many examples can be quoted to show that this rate is conservative.

The rate received on balances placed on time loans may be taken at from 4 to 6%, and on call loans at from 2 to 4 per cent.

Time of Construction.—The assumed periods of inception, right-of-way purchases, active construction, and traffic development should be based on conditions as found on the particular railroad under consideration. On a complicated system, the time required for the acquisition of right of way and for the construction of the more difficult lines should govern, and the assumed date of commencement of active work on the less important lines and branches should be advanced so that sufficient time only will remain for their completion when needed for the initial operation of the system as a whole. Industrial spurs as well as equipment may be assumed to be supplied during the final stage when traffic is being gradually induced. By treating the subject in this manner the building of the various sections and the delivery of equipment will be co-ordinated with a view to completion only as needed, and the undue expenditure of interest on inactive capital thereby will be avoided.

Graphical Analyses.—To assist in determining the lengths of the different stages, and in ascertaining aggregate capital requirements and net interest payments as the work is assumed to progress, graphical

analyses will be found to make this important subject clear and conclusive.

Earnings During Construction.—Proceeding on the theory that the full current net operating income of a railroad will be realized at the end of the last stage of construction, it is, of course, proper that the capital invested should be credited with the net earnings enjoyed during that stage. These may be taken at the full current net earnings of the company for a length of time equivalent to one-half of the educational and development period, except on roads where losses instead of profits may be fairly estimated for the development period. In the latter instances it would seem proper to consider the net losses as legitimate charges to the cost of creating the enterprise.

Working Capital.

That working capital is absolutely necessary for the proper running of a railroad is well shown in the following extract from the opinion of the Chairman of the Public Service Commission of New York, in the Rochester, Corning, Elmira Traction Company case:

"In addition to the foregoing matters, there should be provided upon the commencement of operation a fair and reasonable amount of working capital. The operation of the company can be continued with far greater efficiency, more to the satisfaction of the public, and with better results to the stockholders, if it has at all times in its treasury a working capital sufficient and adequate to meet the requirements of the road. Experience has demonstrated this so many times that insistence upon it or elaborate demonstration of its truth is not required at this time."

Working capital represents money permanently non-productive except through earnings derived from the transportation of passengers and freight. It includes cash held in readiness for the prompt payment of pay-rolls and vouchers, money tied up in bills receivable, and money invested in "stock on hand." The average amount may be ascertained from a study of the annual report of the railroad company.

Depreciation.

Depreciation, modified by the appreciation of those parts that improve with time, should be based on actual conditions and on suitability for continued use. As a rule, several types of depreciable and appreciable property will be found on a railroad.

Bridges, Buildings, etc.—Structures like bridges and buildings, fundamentally sound and good for an indefinite continuing life, may have their depreciation measured by the estimated expenditures required to put them in 100% condition. Generally speaking, obsolescence in such instances is believed to be too speculative for consideration.

Rails.—Certain items, like rails, are good for an indefinite continuing life, during the early stages of which there is a gradual diminishment of value followed for many years by fixed values. Rails usually pass through several stages before they are unfit for further use in track. First, they are utilized on the more important lines until their condition becomes such as to make them unfit for high-speed passenger service. Then they are transferred to lines of minor traffic, or to sidings and yards, where they may remain for an indefinite period. Manifestly, it is improper to attempt to measure their depreciation on an arbitrary mortality basis, as in some instances the character of the traffic may be such as to wear out the rail in one-half or one-third of the assumed ultimate life, and in other cases the traffic may be so light as to permit the retention of the rail in service for possibly twice the assumed ultimate life. Therefore the depreciation of the rail in each instance must be judged by its actual condition and the kind of traffic that it bears. Commercially considered, there are three classes of rail: new, relayer, and scrap. Between the first two there is a gradually decreasing range of value from the new to the relayer price, the difference between them being spreadable over the average period during which it may be assumed that the rail will be suitable for the higher classes of service. Further depreciation below relayer value will depend entirely on actual conditions as the material approaches the point where it is removable as scrap.

Ties and Timber Trestles.—In a number of cases experience has shown that a fairly definite life may be determined. Ties, for instance, from a study of the renewal records of each company, will be found to have an average life, and the average percentage of depreciation of all the ties in track will depend on the time of year when the observations are made. Timber trestles will be found to have average lives, depending on the kind of material of which they are constructed and their location and purpose. In some instances the trestle may be made as good as new by the replacement of a few of the more perishable parts of the structure, and in other cases an entire renewal will be required.

Between these limits, depreciation should be measured by the proportion of the total life of the structure that has expired.

Equipment and Machinery.—With rolling stock, floating equipment, and shop machinery and tools, obsolescence as well as decrepitude should be taken into account, as past experience has proven that there is a well-defined period of economical life beyond which continued use is wasteful or unproductive. In treating this class of depreciation, more than ordinary experience and judgment are required, because of the fact that on the character of up-keep is dependent the economical life of the item that is under consideration. Therefore it is rarely possible to adopt any fixed rule that will be applicable under all conditions.

Graduation and Ballast.—Embankments in the course of time undergo solidification following impact of rolling stock and the action of the elements. This seasoning results in appreciation that may be gauged by the estimated cost of obtaining practically the same desirable condition at the time of the construction of the road. This cost will depend on the availability of water for consolidating purposes, the nature and quantity of material, and the degree to which the act of solidification will interfere with the ordinary manner in which the embankments would be built.

Excavations on a new railroad are known to give much trouble from slides and filling up of ditches, entailing constant labor, often with the use of work trains, until the surfaces of the cuts adapt themselves to conditions, or, in other words, become seasoned. In the case of embankments it is possible to measure appreciation by the equivalent cost of compacting them artificially, but it is not so easy to fix definitely the value of appreciation in excavations; however, a close approximation may be made by estimating the cost of work train and other service for an assumed period of time, taking into account, of course, depths of cuts, their age, and the nature of the material.

Where slopes for fills and cuts in the course of time have become grassed or otherwise covered with vegetation which protects the surface, allowance for appreciation may be made on the basis of the cost of the adding of these items in the course of construction.

For a considerable period after tracks are first ballasted, more or less trouble and expense are involved in preserving line and surface, particularly during periods of wet and frost. This is due to the working of the ballast into the roadbed, the filling of interstices from beneath,

and to the fact that the ties have not become solidly embedded in the ballast. As a consequence of these conditions, extra labor is required constantly, to readjust the track, and there is an absence of the smooth riding of trains which is so essential to fast passenger service and to the minimizing of repairs to track and equipment. As time goes on, the track is raised on new ballast, so as to maintain it at its original height, and the original ballast is pressed or settled downward into the roadbed, where it remains as a firm, semi-porous substratum or blanket to protect and sustain the overlying newer ballast. This newer ballast is the visible portion which appears above the top of the shoulders of the embankment and is usually measured and estimated as ballast. The older or concealed ballast is incorporated in the roadbed, and though it is not measured and estimated as such, its value is reflected in the appreciated worth of the part that is measured and estimated. Experiments have shown that a thickness of 24 in. beneath the bottom of the ties is necessary to distribute the weights of rolling stock properly, and therefore appreciation of ballast may be considered as being measured by the cost of the concealed portion to a maximum depth of 24 in. below the bottom of the tie, less the cost of the roadbed material thereby replaced. Of course, if ballast is dirty and requires cleaning, the approximate cost of forking it over or otherwise cleaning it may be deducted from the appreciation arrived at as above.

Organization of Valuation Corps.

With the general principles settled which are to govern the work, the organization of the valuation corps next commands attention. Viewing the problem in the same spirit that guides the actual creation of a going railroad, it is evident that the mental attitude of the leading members of the corps must be in harmony with the practices that obtain in the inception, construction, and initial operation of the enterprise. Consequently, they must be men of broad experience in their respective fields, capable of adjusting rules and prices to conditions as they find them in each particular instance. Mere inventorying, without the detail and constant supervision of experienced engineers, will produce results which will be easily attackable in court.

In a word, the problem is one that calls for a high order of engineering skill in both rank and file, so that the outcome may reflect all the processes which have a bearing on the final cost of a live railroad.

Valuation of Road Items.—Rather than subdivide the work under a number of engineers having charge of separate subjects, such as grading, bridging, and track, the writer believes that territorial divisions or districts should be established, each in charge of an engineer capable of viewing his assigned territory as a whole, and of personally passing on the value and condition of all items in the light of their relations to each other, except those on which he will require the advice of special experts. In addition, it may be said that territorial valuation has the merit of readily lending itself to the segregation of results so that they may be made available for a variety of purposes, as, for instance, in estimating the cost per ton of transporting commodities from various originating points to destination, and in verifying values used in various taxing districts.

Although engineers with experience and ability in handling the work as thus outlined could personally pass on the majority of the items entering into way and structures, there are several features requiring the attention of special experts.

These consist of right of way and real estate, interlocking and signals, and shop machinery and tools. In addition, the engineer in charge of each district will at times need advice on other specialties, as, for instance, heating, lighting, and power plants in shops and enginehouses, and important or unusual structures.

Valuation of Lands.—The right-of-way and real estate expert should be a man of wide experience in the purchase of land for railroad purposes, having also an intimate knowledge of railroad land values in the section in which he is located. He should be expected to ascertain the current basic values of neighboring lands as a means of measuring the value of the railroad properties, make inquiries of local real estate agents and others familiar with the subject, and, in finally fixing values, take into account the factor in each case that experience has taught should be used as a multiplier in connection with railroad purchases.

It is believed that this method of ascertaining the reproductive cost of right of way and real estate, through the central agency of one having this comprehensive knowledge of the values of land purchased and used for railroad purposes, will avoid the inconsistencies and errors which are common where the work is entrusted to a multitude of local men unversed in the broader aspects of the problem.

Valuation of Equipment.—Rolling stock and floating equipment, as defined by the Interstate Commerce Commission, should each be placed in charge of an engineer of standing in his specialty. Each locomotive, car, and boat need not be viewed personally by the appraiser, but a sufficient number of each class or type may be inspected to fix their value and condition.

Forms and Tables.—The adoption of forms on which observations are to be recorded and results compiled, is, of course, an important step in the work. The forms should be inter-connected in such a way that it will be easy to trace backward from compiled results to the parent underlying data, and they should avoid generalizations which cannot be substantiated by facts as recorded in detail on the ground.

Tables and diagrams are also of value in facilitating field and office calculation of quantities, and as a check on arithmetical work.

Conclusion.

In conclusion, it may be said that the cost of reproduction new appears to be the only measure of physical value that places all railroads on the same plane, and the only one that provides for the inclusion of every element of cost that enters into the creation of a going railroad. If properly applied, it will give due weight to the duplication of lands, under exactly the same conditions as are known to prevail in actual practice; it will provide for the inventorying and pricing of measurable items in such a manner as to recognize their correlation and the varying local conditions that surround them; it will recognize the propriety of adequate charges for overhead costs, including engineering, contingencies, general expenditures, and payments for the use of money during construction, all of which are so essential to the assemblage and endowing with life of the component parts of the system; it will take into consideration net gains or losses during the final or development stage of reproduction; and it will not forget that working capital is as necessary to the success of the enterprise as other capital charges. The fulfilling of these conditions calls for engineering skill and experience of the highest class, not alone in technical fields, but also in the broader domains of administration, finance, and operation.

A physical valuation thus embracing every item of cost that enters into the duplication of facilities in full running order, equipped,

manned, and trained for the same character of service that they are now performing, may be said to include many of the intangible elements of going value which are so difficult, if not impossible, to measure in any other manner. There are, however, other elements of value, such as traffic productivity and operating effectiveness, that cannot be measured in this manner, and therefore are beyond the scope of this paper.

Whether or not physical depreciation should be deducted from the cost of reproduction new, may be said to depend on the purpose of the valuation. If its intent is the ascertainment of a price to be paid by a purchaser to whom is to be transferred the owner's liability as regards future renewals and replacements, then, beyond question, depreciation should be deducted; but, if the amount of capital invested is the point at issue, depreciation, being an element of operating cost and not a wastage of capital, should not be deducted.

DISCUSSION

Mr. Parsons.

MAURICE G. PARSONS, JUN. AM. SOC. C. E. (by letter).—Valuation, to-day, is the fashion, and rightly so, for only with absolute and complete knowledge of facts can justice be done either the consumer or the producer. Surely, this is a great subject, large beyond the ability of any one commission which has not the thorough support of the entire country and cannot rely absolutely on its assistants. The people, the corporations, and the Profession should most heartily welcome Mr. Wilgus' paper as an endeavor to clarify valuation views more fully and to secure the co-operation of all concerned.

It is interesting to note that here, again, the much-discussed subject of depreciation has come up. There is, perhaps, more written on this phase of valuation than on any other, very largely because, like will-power, it is a sort of composite; depreciation is made up, in part, of red lead, stomach, philosophy, bookkeeping, obsolescence, and the methods of treatment throughout. This divergence of methods causes much trouble. Any one of several logical processes will give correct answers to the question of depreciation, although certain steps in one method may differ entirely from similar steps in another.

The author has approached the subject of physical valuation in the large, showing that it is not a question of so many yards of earthwork, so many tons of steel; but that all parts of a railroad must be combined, fused, and vitalized, that they must be made a single working machine. That, however, is not all; the mere inventory of the physical property (if one may broaden the discussion beyond the limit set by the subject) is only a small part of the problem. Before a board can rightly take from one and give to another, it must have knowledge, not only of the physical value, but of the complete economic and financial history of the company. It must know the total cost of producing the commodity (which cost is at times greatly in excess of that of the physical plant) and the total profit derived from its sale. Promoters may embark in hazardous but necessary business during hard times, with the result of large financial loss in the early days; then again there may be absolutely no risk connected with the building up of the business. The development value may be practically nil. On the other hand, regardless of risks which have been taken, subsequent profits, whether great or small, may alter entirely the question of what equity the public has in the business. The real problem begins after the physical valuation is made, and necessitates a complete knowledge of the history of the company and of the financial and political situation during its life. As the physical value must be determined in a broad-gauge manner, so also the entire subject of rate-fixing goes deeper than mere consideration of physical value.

Mr. Parsons.

One question about which the writer has been thinking is: "Where may our theory of valuation lead?" May we not get back to the old labor theory of value, which held, for example, that if a man produced in one day a clock which kept perfect time, whereas another man labored equally hard for a hundred days in producing an inaccurate timepiece, this second man, nevertheless, was entitled to one hundred times as much wages as the first man. We cannot say that a utility is entitled to a fair profit on all the investment unless that investment has been wisely made, and we must judge past actions with leniency. On the other hand, although the best judgment may have been used, it is possible that business has almost entirely disappeared, and that the company is entitled to a rather high rate—to what the traffic will bear. Rates must bear some relation to the actual cash value of the service and to a proper return on judiciously invested capital. The germ of the solution of this question of policy (which allows a fair return on the actual investment which may be considered to measure the labor, and may bring us back to the old labor theory of value) is contained in the case of Smith *vs.* Ames, wherein it is pointed out that the rates as affecting the welfare of the corporation and the welfare of the people must be considered in the light of a complete history of all conditions. It is to be hoped that this general broad subject will be more elaborately discussed, for on that depends the fair rate. Necessary regulation of present-day business gets us into deeper water than the one-time automatic competition of small companies.

As the value of a property depends on the purpose for which the valuation is made, we have several sub-heads of valuation. First of these may be mentioned the physical value, determined by any one of several methods. Next, we may place the development or intangible value, consisting of various items not properly appraisable under the head of physical value, such, for example, as promoter's profit, legitimate costs of franchise, legal and engineering expenses, certain property rights, present value of past deficits, etc. Thirdly, the business value is the net income capitalized at the current rate of interest. The fourth, or going, value is sometimes used synonymously for the development value, or again with business value. It seems, however, that this is in reality a value by itself, differing from those already mentioned, and that it can very properly be defined as the difference between the business value and what the same investment would earn in a bank. In other words, if a capitalist chooses to let his money lie idle—chooses to be unenterprising—he gets a certain amount of interest. If, however, he sees fit to invest this money in a business, to promote a project, guide it safely through the development stage, put it in a good running order, and make the organization as a whole a single living unit—a going concern—he is entitled to more interest;

Mr. Parsons.

he is entitled to a fair return on the investment under the particular conditions which he has met and conquered, and the going value is that excess which the business earns over and above what the same money would earn had he chosen to leave it in a bank. The going value might quite properly be defined as the difference between the business value and the idle value.

Mr. Aldrich.

J. FRANK ALDRICH, ESQ. (by letter).—Mr. Wilgus' paper is the "last word" on this present-day subject, and although the writer will confine his remarks to a discussion of the land element involved, it is difficult to see how he is going to raise many issues in the field covered by this paper; for, after some ten years of experience by valuation engineers and a pretty general discussion of the subject, the author has succeeded in eliminating a vast amount of theory and undigested argument—going directly to the meat of the proposition—and from his record as a practical railroad builder and appraiser has evolved basic principles which are both logical and sound. The writer does not mean by this that the subject is beyond the realms of controversy in detail, but as to the hitherto mooted questions of reproductive value, overhead charges, depreciation, etc., the author is most convincing and in accord with the best authorities on the subject.

As with many propositions of professional interest, the opportunity exists for wide-spread discussion, which is sometimes carried to a point of beclouded mental vision. The sooner the subject reaches the practical stage—where justice to all concerned will be the key-note of discussion—the sooner will there be harmony of action between those who, on the one hand, are honestly striving to do exact justice to the general public, and those who, with equal zeal, represent the railroads.

With the railroads (it has been suggested) this public valuation is to be a "two-edged sword," for, if the result is satisfying as a basis for rate-making, how will it operate when it comes to matters of taxation? The writer does not agree with some authorities: that there cannot be one valuation for the one and another valuation for the other. Bear in mind that these remarks are confined, as closely as practicable, to the element of land values (no inconsiderable part of the whole), and when R. A. Thompson, M. Am. Soc. C. E. (one of the Board of Engineers recently retained by the Interstate Commerce Commission under the Adamson Act), asks: "Is there any logical reason why a valuation for this purpose [the issuance of stocks and bonds] should not also serve—as far as it pertains—as a basis for taxation or for regulating freight rates?" and again: "as far as the State is concerned and to be consistent should not one valuation serve all purposes?" the writer answers, to both queries, emphatically, "no"! If an individual or corporation decides to build a railroad, he, or it, must acquire a right of way, which it is conceded will cost per acre,

Mr. Aldrich. through the country districts, from two to five times the normal acreage values of adjoining land. The fact that it costs this amount more, does not make it worth more, and it should be assessed (for taxation) at what it is worth, not at what it has cost. The reasons for its costing more to the railroad will not be discussed here, for that is now axiomatic and elementary. It is enough to say that it does cost more, as every one who has had broad experience in buying right-of-way property for railroads knows; and it would be a gross injustice, as well as a blunder, to tax the strip of land used for right-of-way purposes at its cost, say, $100 per acre, when the adjoining land is assessed at $25 per acre.

Mr. Thompson cites the Texas valuation, where he says his experience as Appraising Engineer for more than 10 years with the Texas Railroad Commission confirms his belief that, in the absence of actual figures of cost, right of way and other real estate should be appraised at but little in excess of the market value of abutting property, and adds: "the conditions under which the railroads were built in Michigan, Wisconsin, Iowa, and Minnesota cannot have been radically different from those in the southern and western states."

Now, let us see: In the Michigan Appraisal* two illustrations are given to indicate the factor that should be used for reproductive values:

1.—The Pere Marquette Railroad, in Montcalm County, paid an average price of $135.19 per acre, and the 1900 appraisal showed an average of $29 per acre on the 918 acres appraised........Factor, 4.66

2.—The Grand Trunk Railroad paid $491.13 per acre for 63.2 acres, and the 1900 appraisal showed $61.44, or only one-eighth of the actual purchase price..Factor 8

Twenty-three illustrations in fourteen counties showed that the average cost of lands was from 230 to 726% higher than the 1900 appraisal, and this, too, "after the 125% and fixed charges had been added" to the appraisal values.

In Wisconsin, the late W. D. Taylor, M. Am. Soc. C. E., the State Appraiser, said:

"In farming lands, small towns, and suburban and residence property, the right-of-way value was taken to be 250% of the market value for other purposes," and notwithstanding Mr. Thompson's conclusion "that, in the absence of actual figures of cost, right of way and other railroad real estate should be appraised at but little in excess of the market value of abutting property."

Mr. Thompson says: "For rural property, the ratios used by Professor Taylor in the Wisconsin appraisal appear to be quite fair."†

In Minnesota, Appraiser Morgan's findings (after employing several special agents who made a study of transfers and assessed values over the State) were, that in the State at large, exclusive of the St. Paul,

* *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 57.

† *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 205.

Minneapolis, and Duluth Terminals, a multiple of 3 was proper, while in St. Paul he used $1\frac{3}{4}$; in Minneapolis, $1\frac{3}{8}$; and in Duluth, $1\frac{1}{4}$. Mr. Aldrich.

Professor Taylor thought city property should be put in at 133% of market value in strips of 100 ft. width or less, and at 110% where the land was owned in blocks or in width greater than 100 ft. It is evident, however, that no "hard and fast" rule will apply to all parts of the country. Varying conditions appear along different lines, even in the same State. As stated by Henry Earle Riggs, M. Am. Soc. C. E.:*

"It is comparatively simple to fix within very close limits the reproduction cost of tracks, bridges, locomotives, or any of the other elements of physical valuations. Not so with the land."

And again:

"It is impracticable entirely to eliminate differences due to error of judgment on the part of appraisers. Where these differences do occur, experience must be the governing factor."

Were the records complete, an abstract of cost prices to the railroads, together with transfer considerations of adjoining land, and assessed valuations, the problem would be simplified, but, except in isolated cases, such records are not available, and the appraiser must be guided in large part by his or others' experience in the actual purchase of rights of way and other real estate for railroad use.. He must be fortified with ample local testimony as to basic values and the individual knowledge of local real estate agents or railroad employees who have personal information as to actual transactions. When practicable, he should examine recorded transfers at the county seats for a number of years back, and take notes concerning the assessed valuations on properties contiguous to the railroad. All this collateral information, including the names and addresses of local authorities, should be carefully preserved and indexed for ready reference. As Mr. Wilgus indicates—land valuations should be directed from one central head, from one whose experience in acquiring property for railroad use (whether by purchase or through condemnation proceedings) has given him the fundamentals which cannot be ignored in the fixing of reproductive prices.

The appointment of local appraisers has not resulted satisfactorily. Fixed opinions or local prejudices frequently exist among appraisers of good repute, and lack of harmony is bound to show in the returns. A well-drilled force of assistants, organized and directed by the chief appraiser, will produce the best results, and, as the author well says, "avoid the inconsistencies and errors which are common where the work is entrusted to a multitude of local men unversed in the broader aspects of the problem."

* *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 139.

Mr. Aldrich. That problem is: What would it cost to reproduce for railroad purposes the land and other real estate now occupied by a given railroad—other conditions adjacent to and through the country traversed by it being as they are at present?

It is immaterial that the railroad itself may have been in large part responsible for the development which its own operation has brought to the surrounding property. It is inconceivable that the added values which have come to adjoining lands owned by A, B, and C should not be reflected in that owned by the railroad. The assessor certainly thinks it is. If one is in doubt about it, look up the record, for instance, of the increased assessment for 1913 on the terminal lands of the New York Central Railroad Company in New York City.

However, consider for a moment the question of "original cost" of rights of way. Unfortunately, the records of these purchases are not always—in fact are seldom—obtainable. In many cases they are not in existence, so that, to make any practical, general use of such data is out of the question; and, if they were available, their examination and tabulation would involve an amount of labor hardly commensurate with their importance to this subject. It will be interesting, however, to give a few examples which illustrate the elements to be reckoned with by the buying agent when he undertakes the purchase of rights of way. A recent case is that of a road now under construction just over the border line of Canada, which is not dissimilar to some of those on this side of the line. The road in question is to run along the north shore of Lake Ontario—through a fruit belt of much the same character as that traversed by a railroad on the south side of the same lake. The writer was considerably puzzled in fixing the reproductive values along the latter line, for there was but little information available covering the original purchases. The writer's assistant in that work was engaged later in valuing the right of way of the railroad on the north side of the lake and had access to the records of purchase. The twenty-five examples, Table 1, are from that record. The right-of-way agent states that his estimate indicates that the cost of the entire right of way will average between six and eight times the normal value of the adjoining land.

In the examples given in Table 1, improvements and what may be termed consequential damages are alone considered. The names and addresses of owners are known, but are omitted from the table.

If the true values of the land alone were represented in these data, the reproductive value would be represented by the percentages given, which, it will be noted, run from 200 to 800, but the right-of-way agent assures us that the prices allowed for the land alone were fully double their normal value.

TABLE 1.

Mr. Aldrich.

Example No.			Total cost.	Percentage.
1	3.15 acres of land	$200.00		
	Fruit trees	300.00		
	Moving house and shed	350.00		
	(Crossing reserved)		$800	$400
2	Land	300.00		
	2 wells	125.00		
	Moving house and shed	525.00		
	(Crossing reserved)			
	Fruit trees	250.00	1 200	400
3	1.67 acres of land	125.00		
	Angling field	100.00		
	Water cut off	50.00		
	Trees	75.00		
	(Timber reserved)		350	280
4	2.25 acres of land	135.00		
	Drainage, 80 maple trees	215.00		
	Angling farm	100.00		
	(Timber reserved)		450	333
5	1.55 acres of land	116.00		
	Angling farm	100.00		
	Water cut off	125.00		
	Moving fence	9.00		
	(Farm crossing)		350	300
6	3.3 acres of land	247.50		
	Angling farm	100.00		
	Water cut off	150.00		
	In lieu, farm crossing	52.50	550	222
7	1.75 acres of land	131.25		
	Severance from water	125.00		
	Angling farm	100.00		
	Fall wheat	13.75	370	282
8	3.6 acres of land	360.00		
	Angling—4 fields	200.00		
	Water cut off	150.00		
	Shade trees	40.00		
	(1 farm crossing)		750	208
9	1 [illegible] acres of land	150.00		
	Angling and changing fences	150.00		
	Water cut off	150.00	450	300
10	Land	200.00		
	Water cut off	100.00		
	Crossing inconvenience	200.00		
	Rocky land—crossing reserved		500	250
11	5.75 acres of land	550.00		
	40 apple trees	900.00		
	Gravel pit	50.00	1 500	273
12	2 acres of land	200.00		
	House and shop	600.00		
	80 trees	1 600.00	2 500	833
13	Land	100.00		
	10 apple trees	250.00		
	5 plum trees	60.00		
	Moving barn	50.00	460	460
14	Land	100.00		
	Apple trees	280.00	380	380

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TABLE 1.—(*Continued.*)

Example No.			Total cost.	Percentage.
15	House and land	$150.00		
	26 fruit trees	450.00	$600	$400
16	1.72 acres of land	525.00		
	Apple trees	1 080.00		
	Berry patch	395.00	2 000	381
17	3.03 acres of land	600.00		
	Apple trees	900.00	1 500	250
18	1.53 acres of land	200.00		
	Apple trees	450.00		
	In lieu, farm crossing	50.00	700	350
19	1.15 acres of land	150.00		
	Apple trees	250.00		
	Berry bushes	50.00		
	In lieu, farm crossing	50.00	500	333
20	3.17 acres of land	455.00		
	Water	300.00		
	Inconvenience	145.00		
	Crossing railway	100.00	1 000	220
21	1.5 acres of land	210.00		
	Apple trees	175.00		
	Strawberries	150.00		
	Damages	65.00	600	286
22	3.1 acres of land	465.00		
	In lieu, farm crossing	100.00		
	Deep cut through orchard	158.00		
	66 large apple trees	1 452.00		
	25 small apple trees	175.00	2 350	505
23	Land, 3.95 acres	450.00		
	Damages for cutting off water	350.00		
	Damages for crossing track	200.00	1 000	222
24	1.37 acres of land	400.00		
	House	300.00		
	35 apple trees	250.00		
	Well	100.00		
	Removing barn	80.00		
	17 plum and pear trees	70.00	1 200	300
25	0.92 acres of land	200.00		
	27 fruit trees	540.00		
	Strawberries	100.00		
	Damages to building	160.00	1 000	500

TABLE 2.

Town.	Acres.	Cost.	Average cost per acre.
West Haverstraw	22.45	$56 195	$2 500
Stony Point	104.77	187 670	1 800
Highland	105.26	183 985	1 800
Cornwall	35.78	71 530	2 000

Not any of this land is in West Point, which is a Government reservation. This company occupies under a lease from the General Government.

Mr. Aldrich.

It may be said that these are isolated cases, and represent but a small percentage of the entire right of way. This is true in part, but the percentage is not as small as might be supposed, on the other hand, not one of them was a condemnation case. Every one was a purchase, in which an agreement was readily reached between the buyer and the seller. Many owners are holding out for extravagant figures, and condemnation proceedings will follow; commissioners must be paid, and attorneys' fees will add to the expense. The railroads seldom "get the best of it" in cases of this kind, so that we may fairly conclude that the right-of-way agent's estimate of 600 to 800% in this instance fairly indicates the ultimate cost to the company.

A case in point to illustrate how difficult it may be for the appraiser to arrive at a fair value—or rather cost—of right of way, came under the writer's notice in appraising the lands of the West Shore Railroad in 1911. His assistant had gone over the road in advance, and reported an average valuation of the lands between West Haverstraw (north of Haverstraw proper) and Cornwall at from $100 to $300 per acre, basic, and had placed a reproductive value on it at from $300 to $750 per acre. The writer questioned his figures, and went into the matter carefully. Much of this right of way runs along the Hudson River at the foot of precipitous mountain sides—in fact, practically all of that which is in Orange County. Railroad construction there was most difficult. Above West Point there would seem to be but little actual value in the land. Fortunately, the writer was given access to the records of purchase (made in 1886) and was dumbfounded to find the total cost of these lands to be as given in Table 2.

The explanation of the high figures in Table 2 is to be found largely in the consequential damages due to severance, as the river was cut off from large estates by a railroad right of way at grade.

Another example which came under the writer's observation was on the Lehigh Valley Railroad near the Town of Wysox, Pa., where it became necessary to buy a house and lot to gain an additional 20 ft. of width for the right of way. The sum paid for this property was $10 000, and the house with the remainder of the property was sold for $5 000, making the 20-ft. strip (worth normally perhaps $500), cost the Railroad Company ten times its actual value. "Consequential damages," it will be perceived, is no small matter in railroad construction. A whole chapter might be given to this subject in view of the important and growing demand for the elimination of grade crossings. One example alone, which came under the writer's notice, may suffice for illustration. The New York Central Railroad paid, as its share, $3 132.98 and $32 592.87, respectively, for these eliminations at Seneca Street and William Street, in Buffalo. The total awards for these two crossings amounted to $7 531 and $65 185.63, respectively.

Mr. Aldrich. "Franchises and Street Crossings" is another branch of this subject to which much space might be given, but it is not the intention at this time to discuss any of these matters extensively. The writer has tried to give, as briefly as possible, some of the reasons "for the faith that is in him," and has given a few examples which have come to him through experience in this work to justify that faith.

The commonly accepted method of arriving at franchise value, through gross earnings and revenue, will hardly apply to railroad corporations, and it is perhaps of questionable propriety that any account should be taken of corporate franchise when applied to its charter. With this the writer has nothing to do; but, to ignore the element of value which applies to local franchises or rights to occupy or cross streets and highways, or even streams, would hardly be sound. Has the Park Avenue franchise, granted many years ago to the Harlem Railroad for an entrance into New York City, no appraisal value? Has the franchise which permits the New York Central and Hudson River Railroad Company to occupy Washington Street, Syracuse, for a distance of 1.2 miles no value?

The cost of a private right of way through either New York City or Syracuse, at the time these rights were granted, would have been comparatively insignificant. What is the right of way through the heart of Syracuse worth to-day, and how many millions of dollars would it take to purchase a private entrance to 42d Street from the Harlem River (more than 4 miles) if it had to be bought to-day? It might have been acquired for considerably less than $1 000 000 then —could it be duplicated for $25 000 000 now? Is genius and foresight to have no credit for risks of failure taken fifty or more years ago? Should capital invested in this particular line of business be singled out for attack and be deprived of the same natural benefits which come to other investments made in the same period?

The general rule to be followed in arriving at a just value of the space occupied by a railroad, at a street, or highway, or navigable stream crossing, is to multiply the average value of the abutting property on both sides by the area and divide by two, on the theory of joint occupancy by two interests—the public and the railroad. If the crossing be occupied by three interests (whether at grade or otherwise) then the divisor is 3, and so on. The same rule will apply to street or highway occupancy along and through the streets, unless sole occupancy of a given space is enjoyed by the railroad, when, if the right is perpetual, full value is taken.

Rights of way along and through streets, not at grade, should be estimated at one-third the value of the surface land.

These rules are quite general, and have been adopted as the result of careful thought and analysis growing out of study and experience in this feature of the work. All crossings not at grade are attended

by increased cost of construction to the railroads, and, if at grade, the cost of watchmen, or otherwise safeguarding them, puts an added burden on the railroad. If further justification be needed for including a value for public crossings in this estimate, it should be remembered that revenue is based on mileage to a large extent, and that, therefore, the value of every foot of right of way should be properly considered in some logical manner. Mr. Aldrich.

As to what lands, if any, owned by a railroad company are to be eliminated from consideration—in the matter of rate-making, for instance, the appraiser has nothing to do—that is a question for the Interstate Commerce Commission and the railroad company to settle. It is the appraiser's business to appraise all the lands of the company, and, in fixing his value, to differentiate between lands actually in use or needed for the operation of the railroad, and those which have been abandoned, such as depleted gravel pits, parts of original right of way, and even where surplus land has been acquired for future yard extensions.

It may be contended, for instance, that houses erected by the company, or purchased by it, for housing employees in the vicinity of shops or transfer yards, is not real estate proper to be included in a valuation for rate-making or of consideration in the issuance of the company's securities. However that may be, it will not affect the method to be used in its appraisal. It is "outlying" property, in any event. The company would not be confined to one particular location for such facilities, and its reproductive value would be the bare value of the land and the present depreciated (if any) value of the houses. Exhausted gravel beds or abandoned right of way would in most cases have but nominal value. Many similar instances could be mentioned, but common-sense judgment in most instances will govern. Large tracts of land purchased for yards, water supply, etc., are ordinarily acquired at a smaller figure, relatively, than right-of-way strips; on the other hand, "plottage" is an element to be considered, particularly in cities; so that no fixed rule will apply.

The question of land or money donations can be disregarded, for two reasons: First, because property is property always, and whether the railroad company or an individual has acquired lands without monetary consideration, or at a low figure, or as part consideration for the risks of investment, its value is an entity and cannot be ignored, either by the assessor or by the stockholder. Second, the practice of donations is a thing of the past—and we are dealing with the present. As part of the "original cost"—that is another proposition. There is no special rule for appraisal based on population that will apply. This will be disclosed by conferences with real estate men in different parts of the system. The attitude of owners will be found to be widely different in different places, but it will be found to be very nearly axiom-

Mr. Aldrich. atic that the more valuable the land required, the less the size of the factor which should be used to indicate its reproductive cost. This becomes evident when the company negotiates for a right-of-way strip through waste lands. An acre held by an owner here, or a few acres by another owner there, may be worth only $5 or $10 an acre, but, unless the wily farmer can get $50 per acre, it will, as a rule, go to condemnation, where the expenses will surely bring the cost up to the holding price.

Before leaving this subject, the writer should perhaps mention one or two additional examples to illustrate what has preceded. The normal value of land in Jersey City, where one of the shorter branches of the Lehigh Valley Railroad crosses the main line of the Pennsylvania Railroad, is 60 cents per sq. ft., but it cost the former company $3 per sq. ft. to acquire an easement under the latter line (through Court proceedings, if the writer remembers correctly), and later the Pennsylvania Railroad Company paid the same price for a portion of the Lehigh Valley property at the same point.

At another point, in Bayonne, it cost the Lehigh Valley Railroad $1.50 per sq. ft., when the normal values at that point were 25 cents per sq. ft. In Joliet, Ill., the following case came under the writer's observation: The right of way of the Rock Island Road was relocated, and it became necessary to tear down a 5-story hotel building, worth approximately $100 000. An appraiser, going over the line in later years, might have no knowledge of this, and no consideration would be given to it, yet it was a part of the cost to that railroad. In fact, other buildings at this point were either totally obliterated or there were consequential damages amounting to several hundred thousand dollars; and these are by no means isolated cases. Therefore, if the records were in all cases intact, a valuation based on "original cost," would be quite as satisfactory to the railroads as any reproductive valuation that could be made.

There are other elements of interest which enter into a proper consideration of the question. The rather perplexing problem of fixing a fair valuation on docks, piers, and water-front terminals will not be discussed at this time, nor the asset value of the virile railroad organization, which only time and a high order of executive control develops.

The "Physical Valuation of Railroads," as a science, is still in its infancy. The Courts probably appreciate this, and will no doubt be influenced in their rulings by the results which, as time goes on, will be worked out by engineers and students who are giving their time to the solution of these problems.

The Supreme Court decision in the Minnesota Rate Case was announced just as the foregoing had been written, and, after a careful reading, the writer finds no good reason for changing the views which he has just expressed. It does not seem possible that the land features

involved could have been properly presented to the Court. Like others who have discussed this problem, it does not differentiate between the terms "Value" and "Cost." The Court says: Mr. Aldrich.

"It is at once apparent that, so far as the estimate rests upon a supposed compulsory feature of the acquisition, it can not be sustained," and again, "it is also said that this price would be in excess of the present market value of contiguous or similarly situated property," but (it adds) "supposing the railroad to be obliterated and the lands to be held by others, the owner of each parcel would be entitled to receive on its condemnation its fair market value for all its available uses and purposes," and:

"Moreover, it is manifest that an attempt to estimate what would be the actual cost of acquiring the right of way, if the railroad were not there, is to indulge in mere speculation" and "the values of property along its line largely depend upon its existence" and, finally, "the assumption of its nonexistence, and at the same time that the values that rest upon it remain unchanged, is impossible and can not be entertained."

All of these arguments, with the exception perhaps of that contained in the last stanza, have been anticipated, and have been fairly met in the writer's consideration of the subject up to this point. There is force in the proposition that values based on present conditions might not obtain, in the event of the non-existence of the right of way. The argument is that the railroad, having been long established, has created values based on its very existence. Their determination, the Court holds, would be "wholly beyond reach of any process of rational determination." Much might be said on this phase of the question, but, the writer thinks, it is susceptible of demonstration that, as a rule, railroad terminals in cities tend to depreciate the value of contiguous property. Good examples of this may be found in Chicago and Buffalo, where real estate men know that the least desirable—and consequently those which have the lowest values—are those properties which surround the railroad terminals. The writer knows of no city, in fact, where this does not apply.

The location of a railroad right of way through the country is usually determined by natural laws, and the farm values affected by its construction extend, ordinarily, for some distance on each side of it, but farmers state that the effect of the building of the right of way immediately adjacent to their property is to depreciate its value, and testimony of this nature is commonly accepted in condemnation proceedings.

Heretofore, the generally accepted theory has been that reproductive values should be based on the present market value of contiguous property, plus whatever increment experience and available data might indicate as proper to add to bring it up to its actual cost to the company. Subject to any further review of the question by the Court of

Mr. Aldrich. last resort, its recent decision must stand, but this does not preclude testimony as to actual cost. The unfortunate situation is thereby created, however, of making it possible for a company owning a line constructed in recent years to produce and get the benefit of such testimony, while the records of older lines may not be available. Records of land purchases for many of the Southern roads, for example, are said to have been destroyed during the Civil War.

A valuation of these properties, based on market prices of adjoining lands, might and probably would result in great injustice to them. The Court refers to this added increment as one "over all outlays of the carrier and over the values of similar land in the vicinity," and says:

"It is an increment which can not be referred to any known criterion, but must rest on a mere expression of judgment which finds no proper test or standard in the transactions of the business world. It is an increment which in the last analysis must rest on an estimate of the value of the railroad use as compared with other business uses."

Herein, the writer thinks, the Court is again at fault, if it applies the principle to railroad valuations generally. In the important railroad land valuations made by the writer, he has never considered "railroad use" as an element of valuation, nor has he heard that term used in this connection. It is not railroad use, it is not alone value, but it is cost that justifies the use of the added increment in making these appraisals. The latter are not used to cover "hypothetical outlays," as the Court asserts, but are based on sound reasoning born of experience and fortified with enough, if not all, actual cases to justify this conclusion.

Mr. Eaton. J. Shirley Eaton, Esq. (by letter).—Mr. Wilgus' paper is comprehensive and critical, and is probably the most succinct statement of the problem which has been made. Its distinctive value lies in its emphasis on the indirect elements of cost of reproduction, which are the inseparable incidents of any actual work. Amateurish valuation fails at this point: It tends to limit its scope to the inventory of physical things at physical measurements at time of valuation, extending the parts inventoried at arbitrary unit prices. The practical engineer understands the necessity of large allowances for contingencies, for errors of judgment, and for changes of detail in the course of construction. Each section of road built involves its special risks and adjustment to local conditions. From the standpoint of personal experience the writer would reproduce a railroad property, carrying it through all the stages of an actual construction under usual conditions. Not only would he include supervision, accidents, taxes, and administrative expense, but he would make provision for the costs of capital that must be held in suspense during the period of construction, and also for the necessary working capital.

Mr. Eaton.

In assigning the work of valuation by geographical divisions, under consulting expert supervision for right of way, signals, and special construction, the author keeps his controlling factors in co-ordination, and gives due weight to local conditions. In predicating an assumed interest rate of 6% on an allowance composed half of stock and half of bond financing, he ignores the fact that many railroads are financed on bond issues alone.

A distinction between valuations according to the purpose which they are intended to serve, is unfortunate. A "valuation" should always be complete and the same thing, having reference to that entity which is defined by the operating income. The law is properly criticized in its provision for a valuation less depreciation. A railroad, as this paper so well emphasizes, is not an aggregate of parts, but an organism of parts, progressively integrated and evolved into a homogeneous whole to produce an operating income. The physical parts have two factors of efficiency which, as capital, must be maintained unimpaired, namely, what may be called the current efficiency and the life efficiency. Every year of use exhausts a year of the life efficiency, and so far reduces the capital value of the original plant. Income account, through which are made the adjustments to maintain capital unimpaired, must bear the charge of accumulating the offsetting reserves from which this reduction of life efficiencies is restored later. Otherwise, the net earnings are overstated to the extent that they have not made allowance for dissolution of the plant. In theory, a net earning figure can reproduce itself indefinitely out of the existing plant. A plant from which depreciation has been deducted will not permit of the indefinite reproduction of existing earnings, and a valuation based on original cost less depreciation, therefore, does not represent the necessary capital involved for the continuation of operating income.

The author touches lightly on the mooted question of including unearned increment in valuation, although this is the storm center about which most of the controversies will arise. In the same category with unearned increment are those values in the physical property representing donations by the community, state, or nation, and the reinvested surplus out of questionably excessive earnings of earlier years. Nor does he touch on another point sometimes urged—the practice of carrying the unearned increment through income account instead of through profit and loss, and thus establishing a rate of return that shall reflect back on the capitalization permitted, which practice would be distinctly wrong. We are at a time when principles of private property in a simple economic society are being complicated by the intrusion of vast values which are the direct result of social production. The analytic methods of latter-day bookkeeping are tending to

Mr. Eaton. distinguish these values from those created by individual enterprise and effort, and to relate them to the sources from which they spring. The problem of how to make this distinction and how far to carry it, becomes a critical and real issue with railroad property first, only because here are gathered, in orderly array, vast aggregates of property to which formulas are more easily applied than to the miscellaneous array of property held privately. We are not ready to believe that the conditions of railroad investment are so different as to make them an exception in this particular, but the question that is raised first in this connection will doubtless be extended in time to include privately held properties as well as those devoted to public utility.

One general criticism of this paper, which applies to all treatments of this subject, is the author's indisposition to strike at the social principles involved and his inclination to limit the discussion to terms and concrete phases. The law is not an illuminating guide. The principles of valuation are not yet stated or known, and while they will be evolved inductively in the concrete studies which the practical application of the law imposes, nevertheless, it would be advantageous, in discussing the matter, to have a leading thinker like Mr. Wilgus intimate the larger social consequences of the undertaking. With these wider aspects before us, in meeting the miscellaneous questions as they arise, there will be built up, not only a practical plan of valuing railroads, but a body of experience out of which will emerge a restatement and a more specific definition of property.

Mr. Howard. C. P. Howard, M. Am. Soc. C. E. (by letter).—In the text of the so-called "Federal Physical Valuation" law, the word "physical" occurs once in connection with classification of property. It also occurs once in a marginal subheading. The law, however, provides for a complete valuation, including "other values, and elements of value, if any," and the "present value" of "all lands, rights-of-way and terminals."

In this connection, the writer desires to call special attention to the subject of land values; to certain important elements which affect such values; to other elements of great importance which affect them from a railway standpoint; and, very briefly, to the question of whether these latter should be considered as "land" or "intangible" values.

A mere statement of facts as to the existence and importance of some of these elements might appear sufficient, and any argument on the subject as an attempt to demonstrate self-evident truths; nevertheless, in view of a recent Court decision, a thorough consideration of the subject would seem to be in order.

Suggested Definition of Value.—What is the value of a piece of property? The following definition is suggested: "The value or market value of a property is its greatest actual value for any lawful purpose for which it may be used and for which there is a demand."

A piece of ground may have no value for industrial or agricultural purposes, but it may have a high value as a residential site. Another tract may be valuable as a site for a wharf, railway terminal, or something else. Its highest value governs; or, if it has value for more than one purpose and can be used simultaneously for both, the sum of the values will govern—for instance, fertile farming land underlaid by a coal vein. Mr. Howard.

Cost of Right of Way.—Mr. Wilgus calls attention to the items of "severance and other damages, plottage, etc."

A narrow strip of land laid off across the country without regard to existing boundary lines cannot be purchased at the price obtaining for ordinary farm lands. As a general proposition, nobody would sell such a strip, splitting his farm in two, for anything like the price per acre which would apply to the farm as a whole. Considered as an article of commerce, it costs more to produce it. To the ordinary price for land must be added the depreciation in value of the remaining land, due to separation of parts and other damages. The Courts recognize this in provisions for damage to the residue. The law does not allow any general increase of land values to offset "damages to the residue"; if it did, the man with a farm near the railroad, but not touched by it, would receive the full benefit, and have an unfair advantage over his neighbor who would have such increase deducted from his land damages. An increase in price, therefore, is lawful, just, and customary.

When a railroad is needed, and such a strip is judiciously located, the demand for the property is imperative, even at such increased price.

Its increased value is due to its shape, position, continuity, etc., elements quite different and distinct from its original value for farming or other purposes, and having little relation thereto.

Continuity.—The continuity of a railway right of way is undoubtedly a most important element of its value. A right of way along a mountain stream might be extremely valuable for railway purposes if continuous; but if it is blocked by some impassable obstruction at one point, it would have no value whatever. This, of course, is self-evident.

Value for Railway Purposes.—Railroad companies have fought each other "tooth and toe-nail" for gaps and passes through the mountains, and elsewhere. In these cases the value for a lawful purpose and the active demand for same are perfectly evident. Among a number of contests of this sort, which have come under the writer's observation, the following instance may be cited:

Two railroad companies located their lines through a gap in the mountains. A bought the right of way. B claimed prior location and sought to condemn. The lower Court decided for B, and testimony was taken as to the value of the land. An engineer for A, questioned

Mr. Howard. as to its market value, placed it at $25 000, based on value to its owners for the purpose for which they proposed to use it, that is, for the construction of a railway tunnel. Its value for any other purpose would have been a mere trifle. The sequel is interesting. Road A, being deprived of its right of way, went ahead with the construction of a longer tunnel on another line a few hundred feet south of the main gap. Road B completed the tunnel through the main gap. The Superior Court on appeal decided against Road B, and Road A then had two tunnels.

As a matter of fact, it is perfectly evident that the value of land for railroad purposes is a distinct and important element of value. One line may be worth as much or more than another line which cost considerably more to build, its greater value in excess of cost being altogether due to its more favorable location, that is, the greater value of its land for railway purposes. This does not necessarily imply a difference in engineering skill, the first line having first choice in the selection of its route.

Existing railroads in many parts of the country occupy natural or strategic routes which, from the shape of the land and the general configuration of its slopes, are especially adapted to railroad purposes. At present, these routes are of great value, and are likely to become more so in the future.

The importance of passes and defiles in the mountains is well known to engineers; for instance, the New River Gorge, occupied by the Chesapeake and Ohio Railway; the Ohio River below Pittsburgh; the Allegheny, Monongahela, and Youghiogheny Rivers at and above Pittsburgh; and the Susquehanna above and below Harrisburg.

Let any one spread out the sheets of the U. S. Topographical Survey from Pennsylvania to Georgia, note the railroads which have recently been built, and study the situation carefully. He will see how difficult, if not impossible, it will be to get another low-grade railroad line from the Ohio River to Chesapeake Bay, or to any point on the Atlantic Coast, below Norfolk, without using the tracks of an existing railroad. The Potomac, James, and Roanoke are the only rivers which cut through the Blue Ridge Mountains. On the route from Pittsburgh to the Potomac both banks of the Youghiogheny are occupied; this is also true of the Potomac from Piedmont to Cherry Run, near Hagerstown, Md. Between the Potomac and the Chesapeake and Ohio Railway, are a number of high ranges running northeast and southwest, and lapping past each other like interlocked fingers. From the Chesapeake and Ohio, on New River, southward to the Norfolk and Western Railroad, there are numerous ranges of hills and mountains; and from Roanoke south to Rabun Gap, Georgia, the eastern slope of the Blue Ridge falls off like the roof of a house, with the highest mountains

of the Alleghanies to the west, and still farther west lie the Cumberland Mountains. Mr. Howard.

It is impossible to state how many other railroads would have been built from Pittsburgh east, if there had been a place to put them, or how many east and west lines south of the Potomac; one would have to be familiar with all the various surveys of proposed railroads. The writer knows of one company which attempted to build west, in the territory between the Potomac and the Chesapeake and Ohio Railway; it built a short distance and stopped.

The value of ground for railway purposes, or its location value, is well illustrated in the case of terminals in a large city. If the business is sufficient to justify the construction into the city of a new line (B) at a cost of $10 000 000, then, as a matter of fact, the value of the old line (A) entering the city from the same direction and giving the same facilities, would be not less than $10 000 000, even though the cost of reconstructing it on its present location should be estimated at one-half or one-third that amount.

It may be assumed that the old line, having first choice of location, cost less as to physical features of construction, including grading, bridges, etc., that its land cost less, and that at present it would cost less, if valued on the basis of adjacent property without improvements. The new line, however, has to buy property with improvements; then tear them down, and build its own structures. Stated in another way, the value of an existing line is evidently not less than the cost of its reproduction on a new right of way, when sufficient demand exists to justify the construction of an additional line; and as no one will invest without expectation of increase, to this value should be added a reasonable percentage for profit.

When the business of the route will support only one road, any special value due to its advantageous location is not so apparent. But if the business is increasing toward the time when it may no longer be accommodated by the increased number of tracks of the existing road (and it is only a question of time when a second and more expensive line will be required), then the value of the location of the existing road, or of its lands, for railway purposes, may be said to be likewise increasing.

Credit for Location Value.—It has generally been conceded that the owner of property is entitled to its value for any lawful use, such as farming, building, etc. It is not the writer's purpose to suggest who should be credited with the special value of land for railroad purposes—the State or the railroad—but rather to emphasize the fact that this value exists.

How to Estimate the Value of a Location.—Railroad companies, if left to themselves, might settle the value of their lines, including that of location (or that of their lands for railway purposes), by the natural

Mr. Howard. process of competition, provided they did not combine; but if the Government decides to make a complete valuation of railroads for purposes of rate-making, it must consider the various elements which would fix the value according to natural laws of supply and demand, such as the cost of construction, number of available routes, and number of roads, tracks, etc., necessary to handle the business.

If it decides to give a railroad company the credit (or, in some cases, perhaps, the debit) of the value of its location, or the adaptability of its land for railroad purposes, how may such value be determined? This is a difficult problem, yet railroad companies have had to solve it in the past, and, if left to themselves, will have to do so in the future. It takes investigation, time, and money. To consider it as an "intangible" rather than a land value makes the problem no easier, if an attempt is made to ascertain its amount in dollars and cents. It must be remembered also that railroad companies themselves have not asked for this investigation.

The process of determining the value of a location, as practised by railroad companies, is simple enough in a way; but it is expensive and tedious. A company desiring to build through a certain territory and finding the natural route occupied by another road, makes careful surveys and estimates of the cost of building an alternate line. If it costs too much, an endeavor is made to secure trackage rights over the existing line. If it is found necessary to build the new line, then, other things being equal, the excess cost of the new line over the old, actual or computed, may be taken as the value of the location of the old line, or the excess value of its land for railway purposes.

"Location," "Land" or "Intangible" Values.—There are certain cases where the cost of reproduction on a new right of way would seem to be a logical method of determining the value of land for railway purposes—in other words, the cost of reproducing the right of way; there are other cases where this method might be used as a check or side light. It probably makes no difference whether it is considered as "physical," "tangible," or "intangible," provided all existing information is used and every effort made to determine its true or probable value.

Mr. Brinkley. M. H. Brinkley, Assoc. M. Am. Soc. C. E. (by letter).—While the general principles enunciated by the author are interesting, and the paper is a valuable contribution on the subject of valuation, the writer desires to take issue with some of Mr. Wilgus' conclusions.

Since the paper was published, the decision of the U. S. Supreme Court has been made public, and its conclusions have been very far-reaching in regard to the value to be considered for rate-making purposes. It clearly sets forth that rates should be charged on reproduction value less depreciation, or what might be called depreciated reproduction value. A quotation in part is as follows: "When an estimate

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of value is made on the basis of reproduction new, the extent of existing depreciation should be shown and deducted." As the significance of the case was the question of rates, there is no doubt of the consequence of the decision that rates should be based on depreciated reproduction value.

In the following discussion depreciated reproduction value will be called present value.

After a railway is built, renewals do not take place at once, but additions and betterments may be made, the cost of which should be charged to capital account. Starting with the property new, there will be a gradual depreciation, based on the rate of depreciation of the different elements of the property. When it is necessary to renew the items of shortest life, allowance must be made for the expense of replacements.

Assume that from the beginning of operation of the railway, the earnings have been sufficient to pay interest on the investment and create an accrued depreciation fund, which, at the end of each year, is equal to the depreciation below reproduction value. If, however, no fund has actually been created, then before renewals start, the net income is larger than in later years by an amount equal to the average annual renewals of the later period. Advantage may be taken of this by paying larger dividends to stockholders, or the amount may be placed in additions and betterments. The latter is frequently done, for often on railways recently built a large number of improvements take place. In either case, the stockholders gain, and there is what amounts to a repayment of capital invested.

For a property which has never made payments into a depreciation fund, the yearly replacements must come out of earnings. Assume that the railway is composed of a large number of elements of different life, such that after a certain age is reached, the present value remains constantly at the same percentage of reproduction value, due to the fact that replacements are the same in amount each year. This is not a special case, as a large amount of railway mileage approaches this condition very closely. If the property is allowed to earn interest on present value, and the cost of the annual replacement is provided for, the rates charged to the public will be the same as if a depreciation fund had been established at the beginning, being kept approximately equal to the total depreciation and earnings continually allowed on reproduction value. In this argument, it is assumed that the rate of interest on the depreciation fund is the same as that allowed on reproduction and present values. The proof of the above proposition is as follows:

Let i = rate of interest allowed on reproduction or present values;
a = rate of interest on depreciation fund;
R = reproduction value;

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$P =$ present value;

$N =$ annuity required for contribution to a depreciation fund established at the birth of the property, which fund at the end of a certain period will remain equal to the depreciation below reproduction value;

$D =$ value of the depreciation fund;

and $A =$ cost of annual replacements.

Then in the case where earnings had always been allowed on reproduction value and a depreciation fund provided, which after a certain age remains equal to the total depreciation, interest on the reproduction value plus the amount of the annuity must be provided by net earnings, or $iR + N$. As the property is assumed to have reached an age when the annual replacements paid for out of the fund are constant, as well as the depreciation fund, it follows that $aD + N = A$, or $N = A - aD$. Substituting for N its value, the net earnings must amount to $iR + A - aD$. But $R - D = P$, and if $i = a$, the amount to be provided by net earnings becomes $iP + A$. This proves the proposition, as the same amount emerges, which it is contended should be allowed for net earnings where no depreciation fund has been provided. It seems to be clear from the foregoing that if earnings are allowed on the present value, together with a surplus for annual replacements, the property is on the same footing with one which has established a depreciation fund by annuities and is allowed earnings on reproduction value.

If no depreciation fund has been established, it should be assumed that the earnings, which should have contributed to such a fund, have been spent in extra dividends to stockholders or used in additions and betterments, in either of which cases the result is that of a repayment of capital. It is accomplished in the former case by direct repayment and in the latter by increasing the reproduction value of the property without any capital outlay. Besides, the apparent loss in value of the property is very much reduced on any railway more than a few years old by the unearned increment of right-of-way values. The term, right of way, as used, includes real estate and terminal lands. The market value of such land is usually more than the amount originally paid for it, and, in the case of the earlier railways, many times more. With the value of the right of way equal to 20% of the total value, it is easily apparent that, in the case of the earlier railways, the unearned increment of right-of-way values might amount to more than 15% of the value of the property. This, with the appreciation of grading, would be sufficient to take care of the difference between reproduction and present values. There may be some railways of light traffic on which excess earnings in their early life and increase of right-of-way and terminal land values have not been sufficient to balance the de-

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preciation. In such a case, the stockholders have simply made a poor investment.

If provision is made by the railway company for replacements by a replacement fund of such an amount that it will be just wiped out in the years of heaviest replacements, then it would be equitable to allow the company net earnings sufficient for interest on the present value plus interest on the amount of the fund plus an annuity for the replacement fund. The same result would be achieved by allowing net earnings sufficient for interest on the present value, plus an annual payment into the replacement fund, no interest being allowed to accumulate in the replacement fund, being given to the stockholders instead for the use of their capital in the fund. As the sum of the present value and the amount of the fund will always approximate a constant value, the rates paid by the public should be the same general average through a group of years as if calculated on the basis of present value alone. Also, considering the fund as a part of the capital, no additions to capital account need be made for replacements as their cost will be taken out of the fund.

Depreciation is not a wastage of capital if its capital value is repaid, but, as far as the property is concerned, there is a diminution of value. If depreciation is a liability of the stockholders, earnings should only be allowed on the present value, for, if earnings are allowed on reproduction value, depreciation then becomes a liability of the rate-payers.

If earnings are allowed on reproduction value, and normal depreciation is treated as an element in the cost of operation that is covered by the rate, then, if no depreciation fund has been created, rates must be increased as depreciation takes place and replacements increase, since the reproduction value remains the same.

If there is no depreciation fund, and earnings are allowed on the reproduction value, then this becomes the selling value, as the latter is the capitalized value of the income. It has been agreed by the author, however, that the selling value or the capital invested by the purchaser should be based on the present value. It follows that earnings should be based on the present value instead of the reproduction value.

For property which has been allowed to deteriorate below the normal present value, it becomes a liability of the stockholders to place the structures in fair condition to the extent of the difference between the deteriorated value and the normal present value. In this case the stockholders should be penalized by a reduction of interest on investment sufficient to authorize this difference in values in a convenient length of time. During this time, the replacements necessary to bring the property up to normal condition, would be made. Earnings, insufficient in the past to take care of replacements, would be mitigating circumstances.

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The decision of the U. S. Supreme Court in the "Minnesota Rate Case" also covers thoroughly the question of right-of-way values. A quotation in part is as follows:

"The company would certainly have no ground of complaint if it were allowed a value for these lands equal to the fair average market value of similar land in the vicinity, without additions by the use of multipliers, or otherwise, to cover hypothetical outlays. The allowances made for a conjectural cost of acquisition and consequential damages must be disapproved; and in this view we also think it was an error to add to the amount taken as the present value of the lands the further sums calculated on that value, which were embraced in the items of 'engineering, superintendence, legal expenses,' 'contingencies,' and 'interest during construction'."

This decision seems to be fair, both to the railways and to the rate-payers. The right-of-way values for the older railways will be much enhanced over the original cost, and it is not probable that the newer lines will suffer any hardship. The normal increase in land values, consisting of yards, terminals, and right of way, after a railway is built, is usually sufficient to provide for the extra cost of right of way over the market value at the time of construction. Some actual data, comparing original cost with present market value for land values of a complete railway system, would be interesting. However, it should be unnecessary to consider this question if we neglect original cost and consider only reproduction value. The present values of adjoining land have been created on account of the existence of the railway itself, and if the railway was to be built at the present time, the present land values would certainly not obtain. The only exception would be in case another railway existed in the same vicinity, which fact would also have influence in creating land values; but in any town certain land increases in value when another railway is built. Nothing can be known of what the land values would be if the railway did not exist. It seems to be expedient as well as good judgment under the circumstances to allow full market value and no more. The only other alternative is to give some consideration to original cost in the fixing of rates. A percentage for contingencies and interest during construction is evidently superfluous, as it has nothing to do with the present market value of the land.

The writer agrees that the ordinary method of calculating interest during construction is unfair to the company. By this method one-half of the total interest allowance for the time of construction is taken; two-thirds or three-fourths would probably be more nearly correct. If this is done, however, care should be taken that a too liberal allowance for time of construction is not made.

The author's views on depreciation are illuminating, especially on the subject of rails. In addition, it would be well to state that rail

breakage should also be considered, and available statistics on this subject for each railway studied. Mr. Brinkley.

It seems to the writer that depreciation in general should be handled only by an expert. The chiefs of ordinary field parties have not the experience and ability to judge accurately of the condition of structures, and the variations in their opinions are too great. In order to obtain the depreciated value, all structures should be examined by an expert and their condition noted. When the inspection is made, he should have the age of the structure, if such information is in existence. The percentage depreciation can be more intelligently noted if the age is known.

In view of this discussion, conclusions indicate that the recent Supreme Court decision in the Minnesota Rate Case, in regard to the basis for the valuation of railroads, is fair both to the railway companies and to the rate-payers. From the arguments and facts it does not seem possible that any other basis for rates than that of present value with right of way included at market value of adjoining lands, could be made without gross injustice.

Albin G. Nicolaysen, Assoc. M. Am. Soc. C. E. (by letter).—The valuation of common carriers to be made by the Interstate Commerce Commission promises to be of such great importance both to the public and the railroads, and opinions as to the proper principles to be followed vary so greatly, that Mr. Wilgus' paper is most timely. Mr. Nicolaysen.

The author has given a clear presentation of his views, and the force of many of the arguments is admitted, but to the writer it seems that the railroads have generally received a little more than the benefit of the doubt. Thus it seems proper enough that some allowance should be made for solidification of embankments, but to advocate that such allowance shall equal the estimated cost of compacting the embankment as an earthern dam would be compacted at the time of its construction, does seem to be rather arbitrary. It is also certain that the surfaces of cuts become seasoned in time, so that the trouble from slides and filling of ditches is materially less in an old than in a new cut, and that the appreciation of the cuts may bear a close relation to the cost of maintaining the slopes from the time of construction until they become fairly stable; but the appreciation of the cuts is somewhat less, and not equal to the cost of maintaining them during the period of settlement, as some cleaning of ditches will always be required. However, where the difference between the two values is small, it seems to the writer to be reasonable and fair to give the railroads the benefit of the doubt, taking into account the fact that, in some cases, valuation for rate-making purposes may work hardship to innocent investors. The author, however, appears to claim that even an additional allowance, equal to the cost of establishing vegetation by artificial means, should

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be made where the cuts have become covered with vegetation which protects the surface. This claim seems clearly against the fundamental principle that physical value is determined as the cost of reproducing the property in its present condition, under the assumption that the natural topographical conditions along the line are as they were at the time when the road was built, while, in other respects, the conditions are taken as they exist to-day; under this theory Nature may logically be assumed to re-establish vegetation in places where it has established such vegetation in the past.

It appears to the writer that most appraisers would make some allowance in the physical value for interest during construction, cost of promotion, banker's commission on securities (but not discount on bonds), contingencies, and similar items, so that the difference of opinion regarding these items would be confined mostly to the question of the proper allowance to make for each. This the writer does not consider himself qualified to discuss further than to state that it would seem reasonable to assume that an examination of the books of recently built roads would disclose data from which these items could be estimated quite closely.

There remains one fundamental question on which the writer differs absolutely with the author, namely, whether the proper physical value for rate-making purposes should be determined as the cost of reproduction new, or whether a deduction should be made from this value to cover depreciation.

As long as it is agreed that depreciation shall be included in maintenance, and that the renewals shall be charged as operating expenses, it seems clear that the physical value for rate-making purposes is the cost of reproduction new, less depreciation. To illustrate this contention it may be assumed that a certain road was built as a unit. When new, the physical value equalled the cost of reproduction new, since no depreciation had occurred. After the road had been in service for one year, a certain depreciation took place, but this depreciation was a part of the operating expenses for that year, and may be assumed to have been placed in a special depreciation fund. If this has been done regularly, there will exist at the present time a fund made up of yearly appropriations equal to the depreciation for each year less the cost of renewals made in the period considered, and this fund should equal the depreciation on the road as it exists to-day. The road under consideration is clearly entitled to a return on the cost of reproduction new without any deduction for depreciation, but the depreciation fund may logically be assumed to be invested in such a way as to earn interest on itself, and the net earnings from operation need only be large enough to provide for a return on intangible values, if any, and on the cost of reproduction new, less depreciation.

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It is granted that such a fund is rarely established, but this does not alter the case; it simply means that the stockholders have drawn out of the road more than was really available for dividends, and have thus reduced their investments in the property by an amount equal to the depreciation. As long as the property is kept in good condition, no objection can be made to the withdrawal of money which, in a sense, belongs in a depreciation fund, because the average condition on a well-maintained road of any magnitude differs but little from year to year, and it will never be necessary, or even possible, to renew all of the property during one year.

If maintenance expenses are considered in connection with the return on physical value, it will be still clearer that depreciation should be subtracted from the cost of reproduction new, in order to determine the physical value on which a return may be expected. If this is not done, then two roads, A and B, for which the cost of reproduction is identical, will be entitled to the same return on their physical values, although A may be a fairly new road and in excellent condition, while B may be old and run down. Under such conditions the maintenance expenses, and, for that matter, the operating expenses too, would be far heavier on B than on A, and, as the roads are entitled to earn maintenance and operating expenses in addition to a return on the physical value, the result would be that the poor road, B, would be entitled to far larger earnings than the good road, A, which shows clearly enough that something is wrong with the basic assumptions. In fact, as the writer sees it, there is no doubt but that the physical value should be determined as the cost of reproduction new, less depreciation.

In connection with the question of depreciation, it would seem that average depreciation, rather than actual depreciation, should be used in the majority of cases. If an attempt was made to base the value for rate-making purposes partly on the actual depreciated value of the physical property, it would mean that the task of revising the valuation in order to keep it up to date would be almost impossible on account of the multitude of items involved. In addition, the writer believes that the result obtained by assuming that, say, all the untreated pine ties on a fairly large road had a present value equal to 50% of the value new, would be more nearly correct than that arising from an attempt to estimate the depreciation of each individual tie, although a still closer value might be obtained by estimating the average depreciation, bearing in mind that the normal depreciation would be 50% and the minimum and maximum depreciation, say, respectively, 35 and 65 per cent.

If average depreciation instead of actual depreciation was used to determine the physical value, the result would be a tendency toward stability in rates, field and office work would be greatly reduced, and the task of keeping the valuation up to date would be made practical.

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It is admitted, however, that the actual depreciation of buildings and structures should be determined, not only because it is required by law, but also because it will furnish a guide in establishing average depreciation of different classes of structures and in analyzing maintenance expenses, the writer's belief being that average maintenance expenses must be ascertained as a step necessary to the determination of intangible values, if any.

As, theoretically, abandoned property must be paid for out of the depreciation fund, it is obvious that such property as a spur to a mine which has been worked out, should only be given a nominal value. The same would necessarily apply to stretches of roads having such curvatures and grades as to make it impossible to operate trains economically over them under present conditions, to bridges which are too light to carry present rolling stock, etc. This, however, should be definitely settled and understood, else the natural result would be a postponement of a very large part of desirable improvement work along these lines, pending the completion of the valuation.

To the writer it seems that the value for taxation purposes may differ from that for rate-making purposes, but he is not certain that it would be impossible to distribute the several items of value between physical and intangible values in a way that is logical, and, at the same time, leaves the physical value the same, no matter for what purpose it is intended. If it were possible, this plan would be highly desirable as tending to give the valuations proper standing with the general public. It requires, however, that the method to be used in estimating the intangible values be determined, and it is greatly to be hoped that some member of the Society will present a paper dealing with this subject.

Mr. Molitor.

F. A. MOLITOR, M. AM. SOC. C. E. (by letter).—The thorough and masterly manner in which Mr. Wilgus has covered the subject of the physical valuation of railways has left little room for further discussion. As the writer's professional experience, however, has been confined solely to the building of railroads, a few points which occurred to him in reading this valuable contribution on the pressing subject now before the Interstate Commerce Commission and the railways may be of interest. It should be stated, however, that the writer's experience has been in the commercial or practical field, and not in the political or academic work of valuation for taxation or rate-making purposes; therefore, his views on the general subject may be biased.

Although the purpose of the Federal Act for the physical valuation of railroads is not stated specifically in that Act or elsewhere, nevertheless, it appears to be for one or both of the following purposes: (1) For rate-making or regulation purposes; (2) to ascertain the value of the railroad properties with a view of determining their present

physical appraisement for the ultimate purpose of Government ownership. Mr. Molitor.

If either of these views is true, then the "original cost to date" is a mere bookkeeping task with the evident purpose of checking the results obtained by the engineering method of "cost of reproduction new" and "cost of reproduction less depreciation." The "sale method" may be dismissed from consideration, because, as Mr. Wilgus states, it can be used for taxation purposes only and hence is a State and hardly a Federal question.

Mr. Wilgus has stated the difficulties to be encountered in obtaining the "original cost to date," and an accountant or engineer may well shudder at the task of ascertaining in only the most general way the original cost of some of the earlier railroads, many of which have been rebuilt on entirely different alignment and right of way, the original roadbed having been obliterated by the elements and become a part of the landscape and topography. What possible value can such approximation of the original cost of a railroad, with additions and betterments and improvements added to date, be to the Federal Government? Or what standing will it have in Court? In fact, to what can an engineer consistently testify in the "original cost to date" except only what the books of the railroad company show?

It seems to the writer, therefore, that the only admissible method of valuation is the "cost of reproduction new" or "less depreciation," and he agrees with Mr. Wilgus' theory that the proper valuation for rate-making and regulation purposes is the cost of reproduction new, the depreciation, if any, being taken care of by the owners and stockholders as a liability. On the other hand, if the valuation is for the purpose of taking over and purchasing the railroad, then the depreciation should be considered only to the extent required in each railroad under consideration. It is at this point that the experienced railroad engineer may use his judgment. The writer is acquainted with many roads which are maintained in a first-class manner and would not in all fairness have any deduction for depreciation, because their general condition for the purpose of the public is as good as, if not better than, when it was new, as viewed and appraised in a broad and equitable light. Speaking academically, it is admitted, however, that each and every tie, panel, or track, or other detail of construction, is not new, and, therefore, from a percentage standpoint, is not "100 per cent. new." It is here that the fallacies of theoretical tables, based on the life of a construction detail or unit, and purporting to give its money value for each year of its life, are evident, when the railroad as a whole, or as an operating plant for the public benefit, is being considered.

Another railroad which by inspection shows a physical depreciation to the extent that repairs of roadbed, bridges, and buildings are required, may well have a "less depreciation" deducted from its repro-

Mr. Molitor. duction cost to the extent that the engineer may estimate, not, however, by counting the bad ties, but by estimating carefully what is required to place the road in proper condition in order to handle its particular traffic economically, and here it must be remembered that the traffic of the Pennsylvania Railroad requires a higher standard of roadbed efficiency than a short line in an outlying district with a light traffic density. Is this not an example of one of the "elements" entering into railroad valuation in respect to the "less depreciation" item? And is it not the point of view to be taken by the valuation engineer that he must apply different standards to different railroads? The writer can well imagine the inexperienced man acquainted only with the maintenance-of-way standards of the Pennsylvania Railroad, or other Eastern railway, being detailed to value some Southwestern line. Would he not unconsciously estimate his "depreciation" on an Eastern standard and thereby injure the Southwestern stockholders?

The writer has discussed the depreciation method because, since Mr. Wilgus' paper was published, the U. S. Supreme Court has ruled, in the Minnesota cases, that it must be considered, and although the writer has been of the opinion that it should rule only in roads under bad repair, we are now confronted with it in the valuation of the best roads, and it seems that, in considering depreciation broadly and having always in mind the traffic carried by each, and, in so doing, particularly avoiding formulas, we bring ourselves under the Supreme Court's decision.

Passing to the details of cost of "reproduction new," which must be first determined, regardless of depreciation, it seems to the writer that, in a general way, such cost can be estimated. It will be closer, perhaps, than an estimate of construction cost based on a careful final location, but, nevertheless, it is an estimate which, as far as graduation and masonry are concerned, is not nearly as close as the "final" estimate of the actually constructed roadbed. The masonry cannot always be reached to be measured. The excavations for foundations and classes of material thus encountered cannot be seen, and many like quantities of construction cannot be determined at all in the absence of plans and actual records of construction. On the more recently built railroads these may be found, but, to the writer's knowledge, no such records exist on many miles of lines, the construction staff having been dispensed with before the construction facts were recorded.

Sometimes the classification and overhaul of graduation quantities can only be estimated in the crudest possible manner. The fact that cuts are many years old prevents the most experienced eye from judging the original classification. Loose débris has slid down, covering the original roadbed walls, or weather has changed the character or classification of the visible material, giving hardly a hint of the

classification of graduation originally moved, and no hint at all as to the quantities of classified material. In respect to overhaul, the original method of handling the work should be known. What sort of plant was used, whether scrapers, wagons, or cars. Here, again, the engineer, inexperienced with the locality to which he is detailed, may fall into error by assuming the use of a grading plant inconsistent with, and expensive for, the original work. Mr. Molitor.

The writer has mentioned some of the units of construction which cannot merely be inventoried or measured, but must be estimated, and, in consequence of which, the personal equation will enter into the valuation to a large degree, requiring eternal vigilance and care on the part of the valuation engineers, in order that their estimates may not be overthrown by the Courts on the testimony of the experienced and expert engineers of the railroad companies.

Referring to some units of construction which can be measured, engineers are sometimes confronted with the unit prices to be placed on them. Land values are the most important of these, and the writer is in agreement with Mr. Wilgus' expressions in respect to using a stated multiplier against the normal market value to determine generally the cost to the railroads, which is known to be in excess of other land purchases; he is compelled, however, to change the method, but not his original opinion, by the Supreme Court's ruling in the Minnesota Rate Case, where it decided against the use of a formula or factor for determining the cost of lands. This decision leaves only the alternative of using the normal present market value, but if abundant proof is offered of the actual amounts paid in good faith for land by the railroads, the writer cannot but believe that such costs of land will be admitted.

"Engineering," in the writer's opinion, has generally been estimated too low, and as far as his experience shows, has been varied from 5 to 10% of the cost of the railroad proper, depending on the locality and topography, as well as the speed at which construction progressed. In the case of a physical valuation, the construction must necessarily be assumed to have continued without interruption due to lack of funds or other causes, otherwise a wide door is opened for varied and excessive engineering expenses. It seems equitable, however, to make due allowance for such delays if they occurred during the construction of a particular railroad.

"Contingencies" is an item which, as Mr. Wilgus suggests, must be given due consideration, and is dependent on the individual cases presented in the railroad valuation.

Such minor items as insurance, stationery and printing, can be taken care of by a valuation engineer in no better manner than that suggested by Mr. Wilgus, but "law expenses" is an item that cannot readily be estimated by comparison with the like expenses of other rail-

Mr. Molitor. roads, because these expenses vary with each railroad and with the necessary legal work that was or should be performed in connection with railroad construction. As a general thing, the writer is of the opinion that law expenses have not heretofore been estimated as liberally as the facts and known expenses seem to warrant. Some railroads, through the selection of a contested route, or through contractors' difficulties, have experienced heavy charges for legal services during construction. When a railroad is valued for the cost of reproduction new, is it to be assumed that no legal difficulties of this nature had been encountered? If so, a just and equitable valuation for this item has not been made.

"Interest and Commissions," which cover the interest on the money invested in the railroad property presumably until such time as the railroad is completed, in operation, and becomes self-sustaining, and bankers' and underwriters' commissions, etc., are items susceptible of division of opinion and many interpretations, when placing a valuation on railroad property.

In the writer's opinion, a discount on bonds, as near as it may be determined by the probable credit of the railroad being valued, should unquestionably be allowed for, as it is part of the investing public's risk assumed in purchasing the securities of a new company, and the interest on which, therefore, should be paid for by the shipping public in the shape of the rates based on the total valuation of the railroad property, which should thus include the discount on the bonds. On one hand we have a small road with poor credit, the original construction bonds of which sold, say, at 60%, while on the other hand there is a subsidiary line of the Pennsylvania Railroad, for instance, the bonds of which, issued for construction purposes, would probably bring par to the investing public. In the consideration of the commissions and discount item in the valuation of these two extremes, should not this be taken into consideration to the extent of allowing no discount in the case of the Pennsylvania Railroad and 40% for discount to be added to the valuation to cover the legitimate risk taken by the investor in the small railroad company's bonds. The public thereby will pay a very small fraction of the interest on the 40% increase in valuation, in the shape of railroad rates. The only alternative would be that the public guarantee the interest on the bonds issued by the smaller company, so that by such additional credit they would bring par and a price equal to those issued by the larger railroad with its good credit.

The interest during construction, of course, is more readily obtainable by multiplying the estimated period of construction by the prevailing interest rate and dividing by the average period of time in which all the money was used, which, for practical purposes, has been taken as two, on the theory that the money expended during construction has been regularly distributed over that period, so that the average

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interest is one-half of the total amount. It is quite possible, on the other hand, that the money expended during construction has not been so evenly distributed as to permit of this course.

A case is within the writer's experience where the graduation was comparatively light, heavy expenditures were made for right of way, and the rails and fastenings were shipped in the early stage of construction, with sight drafts against delivery, so that something like 75 or 80% of the cost of construction was expended in the early months. The remainder, consisting of bridges and buildings, was built after the track was laid, resulting in the fact that 25% of the payments were not actually made until 18 months after 75% of the expenditures were made.

It is believed, therefore, that there is a wide latitude in the consideration of the "interest and commission item" in which errors and omissions may enter, to the manifest injustice of the investing public, on the one hand, and the shippers, on the other, if rate-making and regulation is based on the valuation. The writer is aware that some authorities have questioned the allowance of interest and commissions in valuation at all. To those he would suggest that, in the establishment of unit prices for contract work of any character or in any locality, there is contained this very item which is a universal trade condition and must be recognized. The contractor is obliged to borrow money from his bank to carry on his contract work, as his monthly estimates are admittedly not sufficient to carry his investment for plant and like purposes during its earlier stages. He pays the highest rate of interest, and he is reimbursed for these interest payments in the shape of a percentage added in his unit prices. In many cases the contractor is obliged to pay commissions, and this is also reflected or added in his bid of unit prices made to the railroad company. Therefore, if the railroad company reimburses the contractor for interest and commissions, although they are hidden in the unit prices, why should not the railroad have added to its cost in the valuation of its property by the authorities the item of "interest and commission" as a transaction between the railroad and its banker instead of a transaction between the railroad company's contractor and his banker?

The writer is pointing out some of the items that have come under his experience in railroad construction, and have been practical ones, with the view of promoting further discussion on Mr. Wilgus' paper from those members of the Society who have had charge of or have been connected with State railroad valuation in the past, so that a broad light may be thrown on the subject for the benefit of those who are charged with the Federal valuation of railroad properties.

In the valuation of equipment, Mr. Wilgus suggests that each unit need not be viewed personally by the expert appraiser. Nevertheless,

Mr. Molitor. it would appear advisable for the appraiser to inspect as many of the units of equipment as it is physically possible for him to do.

The writer's theory is that equipment, motive power especially, is subject to heavy wear and depreciation not always in the ratio of its age, because one locomotive or car of an equal age and original construction may have performed a greater mileage, or may not have been maintained as well as another, and though he believes that depreciation of the roadbed and structures must be regarded in the general way as a single proposition, he feels that the equipment can, by actual inventory, be valued with the depreciation deducted and obtained in a careful and more exact manner. At the same time, the general overhauling and repairs made to equipment should be given the most careful consideration. Though the life of an engine may be taken at a given number of years, nevertheless, this life may be lengthened by systematic and careful general repairs and overhauling, which, in the case of many railroads, has been done. The past history of equipment is that it becomes obsolete rather than worn out. Its physical life is longer than its economic life, and it is not at all improbable that the latter should be taken into consideration in the value of equipment. That is to say, the unit value of a car of 100 000 lb. capacity is greater than one of 40 000 lb. capacity because of the greater operating efficiency of the former.

The writer is in the heartiest accord with Mr. Wilgus in giving due consideration and valuation to the preliminary organization expenses which are necessary incidents to the construction of any railroad. From the writer's knowledge, there have been instances in which these preliminary organization expenses have legitimately run into an amount equal to 20% of the cost of construction, and he believes that where a railroad company can show proof that such preliminary expenses have existed, they should be allowed in the valuation.

In conclusion, the writer recognizes the legal limitations within which much of the physical valuation of railroads has heretofore been undertaken by several of the States. He believes that though much of the information gained will be of value in the Federal valuation, nevertheless, there are many additional items which may well be taken into consideration by the Interstate Commerce Commission. He refers more particularly to the so-called intangible values. Some of the values that have been called intangible are not, in his opinion, at all intangible, and he will close by mentioning one very tangible item of the value of a railroad, with the hope of adding to the discussion further remarks on the subject.

The profile of a railroad has a tangible value and, therefore, should be considered in the physical valuation of any railroad property.

The economy of the operation of a railroad plant depends on a great many items, the principal ones being fuel, labor, and the profile. Labor

rates are becoming practically uniform throughout the country, but the cost of fuel varies considerably. No item entering into the cost of railroad operation has a greater bearing on it than the profile, and, therefore, why should not the railroad, which, by advantages of location or by the efficiency shown in the location surveys in obtaining a lower grade line than another railroad, have this advantage expressed in the valuation? Mr. Molitor.

The writer has knowledge of two railroads connecting two industrial centers. One of these has a grade line so much lower than the other that the "movement expenses" or cost of conducting transportation is not more than one-half that of the other road, not so wisely or fortunately situated. Should not the road of the lower gradient be given consideration in the valuation over the road of heavier grades? If valued for commercial purposes, the low-grade line would assuredly be valued higher than the railroad of a higher grade, because its net earning capacity is greater.

As the profile of the road is a physical feature, it would seem that a valuation could be placed thereon. The writer is not prepared to say at this time just how, but that some equitable basis could be arrived at for determining the profile value of a railroad is quite certain, and it is hoped that this subject at least will bring forth an answer from Mr. Wilgus.

The physical valuation of railroads now being undertaken by the Federal Government affords a great field for engineers in general, and for members of the American Society of Civil Engineers in particular, and it behooves them to study the problem carefully with the view of obtaining uniform methods and equitable results, and for such study no better text has been offered than the paper by Mr. Wilgus.

Halbert P. Gillette, M. Am. Soc. C. E. (by letter).—Appraisers of railways and other public utilities differ radically on many important principles, such, for example, as to the propriety of using present unit prices or prices that are the weighted average during a term of years. The writer has found that most of these differences of opinion spring from differences in hypotheses as to the political status of public utilities. Analysis discloses two distinct hypotheses, or theories, which, though rarely reduced to words, exist in more or less definite form in the minds of appraisers. These two theories may be called: Mr. Gillette.

1. The Agency Theory;
2. The Competitive Theory.

According to the Agency Theory, every public service company is a public agent authorized to render certain kinds of service, and entitled to be recouped for all reasonable expenditures and costs plus a fair profit.

Mr. Gillette. According to the Competitive Theory, every public service company is authorized to render certain kinds of service, and is entitled to collect what the traffic will bear under more or less competitive conditions.

It is needless, perhaps, to point out that, until recent years, American railways were not subjected to regulation according to the Agency Theory, but were "regulated" by the natural laws of commerce. Gradually, however, the Agency Theory of regulation has been evolving, but has not yet attained general application. We still see the Federal Government attempting to apply the Competitive Theory to railways, by forcing the dissolution of so-called "competing lines"; while, at the same time, the most "progressive" States are attempting to apply the Agency Theory. The Public Service Commission of the State of Washington recently authorized the consolidation of certain telephone companies, and scarcely was the consolidation effected before the Attorney General of the United States began action, under the Sherman law, to "unscramble" the consolidated companies.

Two political theories, therefore, are now struggling for supremacy; and it happens that these two theories vitally affect the principles to be applied in the appraisal of railways and other public utilities.

According to the Agency Theory, the "value" of a public utility property is the reasonable actual investment of capital in the property. This includes the investment in the physical property and the investment in the "residual development cost" (the unrecouped deficit in fair return on the investment).

According to the Competitive Theory (when logically applied), the value of the property is the cost of its reproduction at present prices, minus the capitalized difference in annual cost of production with the existing plant and with the most modern plant which would give the same service, plus the capitalized value of the profits derivable from the plant during the remainder of the company's franchise or charter. Briefly stated, this "value" is depreciated value plus franchise or going concern value.

Although there is a large element of injustice in suddenly dropping the Competitive Theory—a theory under which railways have so long operated and under which so many stockholders have invested—the recent history of public regulation makes it quite clear that the Agency Theory is destined to be adopted. In the transition period, however, we shall have a mixture of the two theories—a sort of compromise theory.

The State which has gone furthest toward the adoption of the Agency Theory is Wisconsin. We find the Wisconsin Commission, for example, including the "going value" or development cost (equal to the residual deficit in fair return) as a part of the total "value" for

rate-making purposes. Also, it refuses to allow the cost of pavement laid over water pipes, unless the water company itself has paid for the pavement. Likewise, it averages unit prices over a term of years, and, where construction has been piecemeal, it uses piecemeal costs. Each of these four rulings is directly in accord with the Agency Theory, and directly opposed to rulings by some Commissions and Courts in other States where the Competitive Theory may be said to prevail. Mr. Gillette.

Mr. Wilgus gives powerful reasons favoring the use of cost of reproduction new as the "value" for rate-making purposes. The recent decision of the U. S. Supreme Court in the Minnesota Rate Case is adverse to this contention. The Court rules that depreciated value must be taken as the "value" for rate-making. Unfortunately, the railways involved in the Minnesota Rate Case presented their side so poorly that the Court did not have either adequate evidence or well-prepared argument on many vital points. Thus, there was no evidence as to the development cost of the railways. Again, depreciation was calculated by the State's engineer according to the "straight-line formula", and the railways did not have the acumen to put "the boot on the other foot" by demanding that an annual depreciation fund be calculated on the same basis. Had this been done, it would have been shown to the Court that the current maintenance expenditures were not sufficient to cover both the repairs of parts and the depreciation of entire plant units. Several other important errors were made by the railways in presenting their case, but these two are mentioned merely to indicate that the precedents established in the Minnesota Rate Case should not be taken as final by any means.

The author is entirely right in saying that depreciated value is not a rational basis for rate-making. Under the Agency Theory, a new public utility company starting to-day would be entitled to a fair return on its actual investment, and it would lead to "confusion worse confounded" were a part of that investment "written off" each year because of depreciation. The thing could be done, but no useful purpose would be served by doing it, while, on the contrary, rates of charge for service would fluctuate with an ever-changing depreciated value. There is a good deal of "horse-sense" in the illustration used by an attorney who argued that the price of milk is not a function of the age of the cow, growing less as the cow grows older, until it would be almost given away in the latter days of the cow's life. What the provident owner of a cow does is to charge a price for milk which will give a fair return on the investment (not on the depreciated value) plus enough to cover "operating expenses", including depreciation. Should the cow be purchased by another dairyman, it is true that he would pay for it on a depreciated basis, but the depreciated value would be such that, during the remainder of its life, the same

Mr. Gillette. price for milk would yield both a fair return on the lower price paid for the cow and enough to cover "operating expenses", including the remaining depreciation.

The Agency Theory applied to new railways will unquestionably lead to the use of the actual cost of the property as the rate-making basis, for it is not rational to burden the present, in order to relieve the future, patrons of the road, as would occur if depreciation were "written off" and rates were based on depreciated value. But what shall be said of the application of the Agency Theory to old railways which now, for the first time, come under the actual application of this theory? The writer believes that justice demands the most liberal treatment of public utilities, particularly in this transition period. Certainly, it is not liberal to appraise for rate-making any of the property of a railway at less than its cost new. Moreover, to use the depreciated value for old roads would be to apply a policy that would not be applied to new roads, in rate-making. Of course, it may be contended that the owners may thus be paid twice for accrued depreciation, because they have already taken out in dividends enough to yield a fair return plus enough to cover the accrued depreciation. Even were this so, we should not lose sight of the fact that the present owners were not the original owners; and they had no way of foreseeing the advent of this new theory—the Agency Theory. The writer, however, is confident that a careful analysis of the ledgers of all railway companies, as far back as accounting records are obtainable, will disclose that the residual development cost is sufficient to wipe out all accrued depreciation, and more too. Development cost is not, as commonly supposed, a matter of ancient history; for the truth is that almost every extension of a railway involves development cost (deficit in fair return). No sooner has the average railway "nosed out" of its development period than it makes an addition or improvement which plunges it into another development period, the surplus from the old investment being swallowed by the deficits of the new. Can there be any doubt, for example, that the great, new terminal of the Pennsylvania Railroad, in New York City, is a cause of deficit at present, in so far as that terminal is concerned?

This matter of development cost leads to another phase of the subject of appraisals. Mr. Wilgus says:

"With rare exceptions, it is extremely doubtful that the books and records of the railroads of the United States will be found to be dependable for the purpose of ascertaining present-day fair values. Only in recent years have cost accounts been kept in a uniform and complete manner, and even then the almost universal tendency has been to understate charges to construction, and to additions and betterments."

The writer does not agree with the author, either that correct railway accounting is of recent origin or that the records are not depend-

able as a guide for appraisal purposes. He recognizes that old records are not always complete, but the gaps are surprisingly few in most cases, and not difficult to fill by estimates which cannot possibly alter the totals by very appreciable amounts. The railways of America would err seriously were they not to conduct the most thorough analysis of all their old records. Their first great error would be their failure to demonstrate what their development costs have been. Development cost (that is, deficit in fair return) cannot be proved satisfactorily except from accounting records. Their second great error would occur through failure to find all the elements of physical value created by necessary expenditures which cannot be seen with the eye. This, it may be said, can be covered by adequate allowance for "contingencies"; but, who knows what is adequate? The writer had occasion to appraise a large property which had been previously appraised by an engineer who had added 10% for contingencies, but whose appraisal still fell short, by 20%, of the actual cost of the property. Failure to hunt for and analyze the accounting records was at the bottom of this serious underestimate. The company was a consolidation of more than a dozen smaller companies, and the earliest accounting records were nearly 25 years old. Were the earliest of these records valueless in a present-day appraisal? Some, it is true, were of minor worth, but they covered minor elements of value. Nearly all the records served to throw light on elements of present-day value which, otherwise, would not have been discovered.

Mr. Gillette.

In this connection, it may be added that the maintenance records should also be analyzed, for it is true, as the author says, that perfectly correct accounting has not invariably existed. Minor expenditures, which should have been charged to property account, have not infrequently been charged to maintenance, and the sum of these minor expenditures thus charged may be quite considerable. Conversely, abandoned or superseded property has not always been charged to maintenance, but occasionally has remained in the property account. This is particularly true of abandoned railway roadbed. A careful analysis of the construction and maintenance accounts, guided by the appraisal of physical property now visible, will result invariably in the discovery of values which otherwise would not appear in the appraisal. Not only will additional quantities be thus found, but larger unit prices will be deduced; and in this connection it may be well to add that unit prices should not be those obtained under rather favorable conditions, but under average conditions. A favorable condition, for example, would be "summer work". Winter work is an unfavorable condition, but one which is often imperative in railway construction, in order to keep down the interest and development cost. The history of the Northern Pacific extension to the Pacific Coast shows that at times

Mr. Gillette. almost as many men were engaged in shoveling snow as in shoveling earth. It was such conditions as this, coupled with other unforeseen difficulties, and followed by a much greater development cost (deficit in fair return) than was anticipated, which threw the Northern Pacific into bankruptcy not long after its completion to the Pacific Coast.

In recent appraisals made by the writer, it has been his practice to allow 5% for contingencies where the construction ledgers were all available, but where the original records of physical quantities were rather meager. If both ledgers and engineering records of quantities are fairly complete, the allowance for contingencies may safely be eliminated, provided liberal unit prices are used. It should be remembered that a contingency factor is automatically introduced whenever liberal prices are adopted.

Regarding brokerage fee, the writer agrees with the author in its inclusion as a part of the cost; but, of course, the same end may be obtained if the rate of fair return is made high enough to allow for all the discount on bonds, including the brokerage fee. This last view was that taken by the Washington Railroad Commission when the writer was its chief engineer, and it is the view now taken by most of the State commissions.

Interest during construction should certainly be calculated on all land as well as on other physical property. The writer was not in direct charge of the appraisal of the land for the Washington Railroad Commission, and, therefore, did not calculate any of the overhead charges on it. In fact, no overhead charges on the land appear in the final land values fixed by the Commission. It is possible that overhead costs on land were omitted, or else that they were included in the "right-of-way multiples". The writer calls attention to this particular case for two reasons: First, because he believes it to be a mistake not to put the entire appraisal of all railway property under the direction of the engineer in charge. Second, because the writer's total overhead charges on the plant, exclusive of land, have often been divided by the sum total of plant, inclusive of land, giving an erroneous result when taken to indicate the total percentage allowed by him for overhead charges. The author of one book on appraisals and many writers of articles have unwittingly been led into this error.

This is not the only kind of error which has been committed by writers in comparing the percentages for overhead costs allowed by different appraisers. For example, one appraiser may estimate 5% for business management, and another would include this item in the unit price and, therefore, not show business management as a separate overhead cost. Many railway construction contracts have been entered into with a managerial contractor on a "cost plus 5%" basis. The recorded contract prices are then sub-contract prices. An appraiser

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may either use these sub-contract prices and subsequently add 5% as a business management item among other overhead costs, or he may increase all the sub-contract prices by 5%, calling them contract prices, and show nothing separately for this business management. One needs to know the method used by an appraiser in arriving at his unit prices before one can interpret correctly his estimates of overhead costs. Not a few appraisers "bury" many of the overhead costs, leaving but a part to appear specifically as such in the final summary. The present practice of the writer is to show all overhead costs separately. They commonly total fully 25% of the cost of the "unloaded" property.

As previously stated, the Washington Railroad Commission restricted its engineer to a consideration of plant value only, exclusive of land. Hence, working cash capital, brokerage fee, and development cost did not appear among the items estimated by the writer. The Commission listened to more or less testimony by the railways on "going concern value", etc., but announced its conclusions without stating any definite theory as to non-physical values. This failure to delegate to its engineer the study of all elements of value is one which the writer regrets exceedingly. The railways were equally mistaken in thinking that a determination of non-physical value was not an engineering problem; consequently, they did not make a satisfactory presentation of their case as to non-physical values. In all subsequent appraisals, the writer has insisted on having full charge of the entire appraisal, including determination of development cost, land values, analysis of accounting records, etc. Some one with technical training should always be placed in full control of any appraisal which is to be used as a basis for rate-making. If an example were needed to illustrate this point, it could be found in the Minnesota Rate Case, where the railway companies erred so seriously through not having the entire technical part of their case directed by engineers. The natural tendency of a railway company is to ask each department to prepare its part of the data, and the result is a more or less confused mass of facts—often facts which conflict because of their incompleteness or because of the incorrect use of terms. In the Minnesota Rate Case, both "right-of-way multiples" and "overhead charges" on land were thrown out by the U. S. Supreme Court. The inconsistent use of words and lack of proper definitions of terms account as much as anything for this erroneous ruling, for the Court was attempting to use symbols which it did not fully understand, and did not understand because they were not clearly and properly defined by any witness.

It would seem that attorneys would always see to it that at least the definitions of technical terms would be precise, and that witnesses would adhere to such definitions; but attorneys themselves are often lost when it comes to the use of engineering and other technical lan-

Mr. Gillette. guage; so that, if a technical rate case is to be presented properly to a commission or a Court, it should be presented by engineers and by those who have been taught by engineers to use technical terms properly and always with the same significance.

When railway appraisals first came to be used as a basis for rate regulation, all, except engineers, regarded an appraisal as being somewhat analogous to a merchant's inventory of stock on hand—a very simple, though often laborious, process. Gradually it has become evident, even to non-technical men, that an appraisal for rate-making purposes is exceedingly technical and complex. When it is realized, also, that rate-making based on cost is even more technical than appraising a property, we shall have an end to the "hot air" testimony of rate experts who are experts only in fixing rates "as high as the traffic will bear."

Appraisal and rate engineering has already become one of the many branches of engineering. The engineering specialist in this line should be primarily a logician, skilled in the use of language and in the science of reasoning. He should be thoroughly acquainted with the general principles of economics and particularly with the principles of engineering economics. He should be well informed as to the decisions of State and Federal rate-regulating commissions, as well as Court decisions bearing on valuations and rates. He should be personally acquainted with specialists in many lines, so that he may select men competent to give any desired information. He should be thoroughly grounded, not only in the principles of accounting, but in the mechanical details of public utility accounting. He should be an incessant student of the new phases of his specialty and of unit costs of construction and operation. Executive ability is also essential to him, but need not be of as high an order as that required of one who is constantly directing large enterprises. It is needless, perhaps, to add that his character should be such that he would make an impartial judge. Obviously, no man can attain the ideal in this, or in any other, branch of engineering; but, at least, those who employ appraisal and rate engineers should aim to secure men who are idealists rather than opportunists, for this is not a profession where mere advocates will survive.

Mr. Willoughby. J. E. Willoughby, M. Am. Soc. C. E. (by letter).—The writer is in accord with Mr. Wilgus in his conclusion "that the cost of reproduction new appears to be the only measure of physical value that places all railroads on the same plane, and the only one that provides for the inclusion of every element of cost that enters into the creation of a going railroad," but cannot agree that he has taken the correct view of depreciation.

It is true that in a well-maintained railroad the accrued depreciation in track and structures does not lessen its capacity as a carrier to perform the work for which it was created, nevertheless, the monetary value of a railroad with track and structures representing 55% of the service that would be obtained from a railroad with all its parts new (all other conditions being unchanged), is less by the amount of the depreciation. Of the capital originally placed in the track and structures, 45% has been consumed. The replacement of this capital can be effected practically only by the creation of a depreciation accounting with a credit which at all times will be equal to the accrued depreciation. The depreciation fund is properly accumulated from year to year as an operating expense, and is to be regarded as an item of value (just as working capital is) which enters into the total value of the railway property whether that value is being ascertained for the purposes of sale or of rate-making. The value of the physical features of any railway property at any time is: Mr. Willoughby.

The total cost of reproduction new, less the accrued depreciation on those parts so affected, plus the accrued appreciation on those parts so affected, plus the amount of the fund provided for the renewal of the parts on which depreciation has accrued.

In many of the existing railways a depreciation fund, if set up now in the accounting, would cause to be shown a profit and loss deficit which would properly reflect itself in a deduction from the market value of the stocks. The meaning would be that, in the past years of the railway's operation, a definite portion of the capital originally put into the track and structures has been consumed instead of being charged out annually as an operating expense. Certainly, the same arguments which caused a depreciation accounting to be set up for rolling stock, apply to the track and structures.

It has been advocated that inasmuch as the annual repairs and renewals, a part of the annual operating expenses, are sufficient to replace the amount of the annual depreciation, a depreciation accounting is useless. Such a view ignores the capital consumed during the early years of the railroad's operation, when the annual repairs and renewals did not replace the annual depreciation.

There is unanimity of opinion that the accrued appreciation in the roadway and right of way should be taken into consideration in fixing the cost of reproduction new. When that is done, it is proper to consider also the depreciation.

The writer believes that obsolescence should be considered as a form of depreciation, because depreciation, in the sense used in connection with the re-valuation work, covers any deterioration of the part affected from any cause by which that part has a less capacity to perform the work for which it was created.

Mr. Whinery. S. WHINERY, M. AM. SOC. C. E. (by letter).—The author refers to the widely different opinions and practices of engineers, appraisers, and the Courts in the valuation of land used for right-of-way and other railroad purposes. Such differences are doubtless largely due to the fact that the purpose for which an appraisal is made controls the point of view to a greater extent than in any other class of railroad property. This fact seems not to have been generally recognized and given proper weight in appraisals and in discussions of the subject, particularly by the public.

The importance of the fact that the purpose for which an appraisal is made must control the methods and values used, was early impressed on the Commission appointed in 1909 to re-appraise the railroads and canals of the State of New Jersey.

The Joint Resolution under which the work was undertaken, required that the Commission should prepare and report a "true and complete inventory and appraisal of the true value" of the railroad and canal property of the State, and, further, though rather incidentally, stipulated that the valuation "shall be in a form available for the purpose of taxation under existing laws".

In their first (progress) report, the Commissioners stated:

"It is not positively stated in the Joint Resolution that the valuation is to be solely for taxation purposes. A true and complete inventory and appraisal of value seems to be called for regardless of the purpose for which the valuation is made. But the term 'True Value' is not definite, since the value of any property varies with the standard of value applied and the point of view from which it is judged. From the very nature of railroad property and the manner in which it is held its market value, in the sense in which that term is applied to other property, is difficult to determine. The commercial value of a railroad is usually determined by its net earnings and the returns made to its owners, quite independent of its cost or of the value of the property it possesses. Frequently there appears to be no close relation between the cost of a railroad and its commercial value. Valuations for the purpose of fixing rates, or for determining proper rental, or terms for joint running operation may also be quite apart from the commercial or the intrinsic value. The value of the franchises and the other intangible property of a railroad may be differently appraised according to the purpose of, or the point of view from which the appraisement is made. Finally, the value of this class of property for taxation purposes may be quite different from any of those named above. It seems, therefore, impracticable to apply any single valuation of the railroad and canal property of the State to more than one specific use, and if this be true, it follows that the purpose of the valuation should be determined upon before an appraisal is made.

"In view of all these conditions, and after a careful consideration of the Joint Resolution, and in view of the necessity of establishing the particular kind of value we are called upon to determine, your Board has thought it best to appraise the various railroad and canal

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properties from the viewpoint of the use of the appraisal mainly for the purposes of taxation."

From the wording of the Resolution, it may fairly be inferred that although the members of the Legislature had in mind the use of the valuation chiefly for purposes of taxation, they were under the impression that the phrase, "a true and complete inventory and appraisal of the true value of railroad property", could be applied to any purpose for which the value of the railroad and canal property of the State might be needed.

There is abundant evidence that the same idea is held by a majority of the public, including members of Legislatures and of Congress. It is important for many reasons that the public mind should be disabused of such erroneous conceptions, and it should be one aim of public discussions of the subject to educate the public on this point.

In fact, there is great need that the public should be educated on the whole subject of the valuation of quasi-public property, and for this reason it would be well that technical discussions of the subject should be sufficiently elementary, and couched in such language that any intelligent citizen could read them understandingly; and that a greater effort should be made to give these discussions as much publicity as possible.

The question of the proper valuation of railroad real estate is complicated and made more difficult by the widely varying elements which enter into the original cost of the property and the conditions under which it was acquired and is held by the railroads. These elements are of such character and of such magnitude that the ordinary standards of fair market price are usually only partly applicable in determining the value of the right of way, either at the time it is acquired or later. Even where the cost of the property to the railroad is ascertainable, it is not always a correct basis of valuation for some of the purposes for which an appraisal may be required.

In discussing the subject, it is the purpose of the writer to deal chiefly with broad general principles, but to consider these somewhat more in detail than has been usual, and to review the various elements of cost and value that enter into an appraisal, for any specific purpose, of right-of-way land. It seems to be necessary, in the interest of clear statement and argument, to recount facts and principles which are neither new nor novel, and the discussion, therefore, may be somewhat tedious to the reader. The writer will attempt to deal with the matter from what he conceives to be a sound and reasonable business standpoint, with little reference to statutes and Court decisions, which may vary in different States. In any specific case, the laws and Court rulings controlling in that locality must, of course, be complied with,

Mr. Whinery. but sound principles of justice and equity must be the fundamental basis of all appraisal work.

What will be said refers more particularly to right of way proper, rather than to lands acquired and held by railroads for terminals and other improvements. The status and basis of valuation of these latter may, and usually do, require special consideration and treatment.

Right-of-way land is held by the railroads under one of two forms of tenure: ownership in fee simple, or some form of easement. In the former, absolute title to the property is conveyed to the railroad; in the latter, only certain rights to occupy and use the property for a specified time or purpose are acquired. The distinction is clear enough in theory, but not always in practice. A conveyance in fee simple may contain conditions, as a part of the consideration, which may cause the property to revert to the grantor, or which may be a continual menace to the title to the extent that it amounts to little more than an easement; and an unconditioned, perpetual easement may be equivalent, for practical purposes, to ownership in fee simple. The tenure under which the land is held by the railroad, therefore, is a matter to be given due weight in its appraisal; but, for the purposes of outlining general principles in this discussion, it will be sufficient to consider only right-of-way land owned in fee simple by the railroad.

Into the consideration, or price paid for a typical right of way, however that price may be determined, three principal elements usually enter:

1. The fair market value of the land.
2. Permanent damages to the remaining property from which the right of way is severed.
3. Temporary damages.

To the sum of these items must be added, to find the actual cost to the railroad:

4. Expenses incidental to the acquirement of the property.

Permanent damages are such as permanently impair the value of the property, a part of which is taken, for the uses to which it is, at the time of the transaction or may be in the future, devoted. The cutting of a farm into two parts, separated by a railroad, involving permanent inconvenience, danger, and increased cost of improvements and of operation, interference with drainage, or with satisfactory sub-division, is an illustration of permanent damage. Temporary damages are such as entail the expenditure of money or cause temporary inconvenience and loss, but do not of themselves impair the present or future fair market value of the remaining property. The cost of moving a building located on the right of way to a new site where its value, convenience, and usefulness to the owner

are not decreased, or the cost of removing or rebuilding fences and other improvements, are fair illustrations of temporary damages. Mr. Whinery.

While these elements of market value, permanent and temporary damages, may not be itemized separately in the settlement or award, it is safe to say that they enter into the aggregate consideration paid for nearly every right of way acquired. The necessity for recognizing and distinguishing between them in some appraisals will appear subsequently in this discussion.

The fourth element, incidental expenses incurred in obtaining a right of way through a property, is as truly a part of the actual cost to the railroad as the nominal price paid to the owner. This would seem to be such an obvious fact as to require no argument, and yet it has been denied or ignored in some Court decisions and in a number of appraisals of cost of reproduction. It includes such items as cost of conferences and negotiations, legal services, cost of condemnation proceedings, etc.

The compensation or consideration paid for a right of way is determined either by mutual agreement or by the award of a theoretically disinterested body of men, on the evidence presented, in the judicial procedure known as condemnation. In case of settlement by mutual agreement between the railroad and the owner of the property, the amount of consideration may vary nominally from zero (donation) to any figure which the railroad may be willing to pay and the owner to accept. The compensation thus agreed on, however, cannot always be taken as a just measure of the cost or value of the property transferred. There may be other valid considerations, which may or may not be named in the deed, but, whatever they are, they do not affect the actual value and, generally, not the ultimate cost of the property to the railroad or its appraisal value for some purposes. This is true even where the right of way is donated. If one friend is good enough to donate to another \$1 000 to be used in establishing a new business, and the money is thus used, it becomes, legally and morally, as truly a part of the latter's invested capital as the money supplied by himself. As a matter of fact, however, a donation of right of way is practically almost never a donation except in name. The so-called donor expects to, and usually does, in some way, get value received, and the railroad pays a price, usually in the form of supplying service for a time at a loss, due to the building and operating of its road in advance of the time when the developed business becomes sufficient to yield adequate returns. It may safely be asserted that in such mutual settlement a fair and proper consideration passes, directly or indirectly, between the grantee and grantor, and that the full value or ordinary cost of the donated right of way should be reckoned as a part of the money invested in the road.

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On the other hand, if the railroad agrees or is compelled to pay an exorbitant price for the right of way, it must be assumed, unless fraud or gross misconduct can be shown, that the money so paid represents as truly a part of the legitimate and necessary cost of the road as though it had been expended for rails and cross-ties. This is true whether the sum paid was a reasonable price for the property or included excessive actual or alleged damages, or was consented to by the railroad to avoid delay or litigation. Where the right of way is secured by condemnation, the theory is, and the fact must be assumed, that the compensation awarded was determined by a body of competent and disinterested men after weighing all the facts, and that the sum awarded was fair and just, and is, therefore, a proper charge against the cost of the road and a like credit to its capital account.

An appraisal of the value of the physical property of a railroad may be undertaken for a number of purposes, among which may be named:

1. To ascertain the cost of the railroad for the purpose of determining the reasonableness of its capitalization;
2. To ascertain the value or cost of the railroad property as an element in the determination of proper and reasonable rates;
3. To determine the value of the property preliminary to a prospective purchase or sale of the railroad;
4. To ascertain the value of the railroad property for the purposes of taxation.

Confining the discussion to right of way and real estate, we may consider the principles and practice that should control in an appraisal for each of these purposes, or cases.

Case 1.—Appraisals to Determine the Reasonableness of Capitalization.—The word "capital" is variously defined. Applied to railroads, it is generally understood to mean the amount of outstanding evidences of cost or debt in the form of stock and bonds. The scientific as well as the ordinary and common-sense definition, however, is that capital is the amount of money permanently invested in a business, and the word is used in that sense in this discussion. It is a reasonable proposition that the evidences of indebtedness of an enterprise—stocks and bonds—should not much exceed the amount of money actually invested in the business, which, in the case of a railroad, may be defined as the amount of money expended originally in its organization, financing, construction, and equipment, plus the amounts since expended in additions and betterments, plus a reasonable amount of free money required for conducting the business, commonly called working capital; and the object of an appraisal, with reference to capitalization, is to determine whether such equality exists.

From this point of view, the question of capitalization is essentially one for the accountant rather than the appraiser. Under usual or normal conditions, neither the present reproduction cost nor the market value of the property can be relied on to disclose the amount of capital which has been actually and legitimately invested in the railroad. Thus, it is possible to conceive of two roads which, to-day, are identical in every respect. One of them, however, was built a generation ago and the other quite recently. When the first was built, the prices of construction work and equipment were materially lower than when the second was built, and the capitalization of the latter will necessarily be correspondingly higher. It would be obviously unjust to assume, because their property now inventories the same, that the younger road is over-capitalized. Mr. Whinery.

The real question to be answered is: What amount of money has been, and, therefore, is now, actually invested in the property? In other words, it is a question of cost and not of present value. The principle is here stated in its broadest terms, and certain reservations must be kept in mind. The permanent destruction or abandonment of property, the payment of dividends out of capital or out of earnings necessary to maintain the property in normal condition, and other like practices which are, if not dishonest, violations of sound business principles, may impair the capital, and if found to exist, must be taken into consideration. On the other hand, the retirement of bonds by paying them off, though it may reduce the outstanding evidences of indebtedness, commonly called capitalization, does not decrease the actual capital or money invested in the road.

Unfortunately, correct and adequate accounts of expenditures chargeable to capital account from the beginning are usually not available. Either they are wholly absent for early periods, or are incomplete, or have not been kept in such a way as to distinguish clearly between betterments on the one hand and maintenance and operation on the other, and it is impossible to determine from them the true investment cost. The very common practice of charging extensive betterments to cost of operation or maintenance, which has prevailed to a large extent in the past, is a pertinent illustration of this condition.

In the absence of complete and trustworthy accounts, an appraisal of the property may be resorted to as the best way to ascertain an approximate estimate of the amount of money invested, or, in other words, the reasonable capitalization.

If the principles here laid down are sound, it must be obvious that an appraisal for this purpose should aim to disclose the actual cost of the property and not its present reproductive cost or marketable value. An inventory of existing property is, of course, necessary, but the unit prices applied to determine aggregate value should be those of

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original cost and cost of betterments at the time they were made. Neither depreciation nor appreciation of original values has any proper place in such an appraisal, unless it appears that the capital has been impaired by some of the actions or causes already mentioned.

The depreciation of the property by ordinary causes (excluding those named which obviously impair the capital) does not decrease the amount of money invested, nor does it ordinarily decrease the capacity of the property for rendering service, as long as the property is kept in reasonable operating repair. A locomotive, for instance, may be ten years old, and yet be as capable of efficient and economical service for the work required of it as when it was new. Even when so worn out as to be incapable of satisfactory service, it will be replaced by a new one, which, if purchased at the same price as the old one, will neither increase nor decrease a properly kept capital account. Nor will any depreciated value assigned to it at any time affect the amount of money invested, on which interest and dividend charges must be paid.

On the other hand, any appreciation of the value of a property, by causes other than the actual investment of additional capital, does not affect the capital account. It is a so-called unearned increment which properly belongs in a surplus rather than a capital account. Nor should earning capacity represented by the market price of stocks or bonds be given any weight.

Applying these principles to the appraisal of right of way and real estate, it follows that the object should be to ascertain and use values which represent the actual cost of the property to the railroad. In such an estimate of the value of right of way all the elements which go to make up the total cost, fair market value, permanent and temporary damages, and incidental expenses, should be reckoned and included. Practically, this may ordinarily be accomplished most satisfactorily by applying a properly determined ratio or multiplier to the fair local market value of similar land at the time the property was acquired. Usually, sufficient records of actual cases will be available to establish, with reasonably close approximation, the value of the multiplier to be used. This matter will be considered more at length under Case 4.

Case 2.—An appraisal made for the purpose of establishing or regulating rates should be governed by the same general principles and methods as one made to determine the reasonableness of capitalization. In fact, their ultimate object and uses are substantially the same.

The lowest rates which a railroad may reasonably or legally be required to charge are such as will repay the total cost of the service rendered, including interest charges and a fair return on the money invested, as any lower rates would be confiscatory. Interest and dividends are a part of that total cost, and as they are functions of the capital invested, the latter must be known in order to determine legitimate cost and to frame proper rates.

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This is the only logical theory on which an appraisal of physical property can be called for or utilized as a basis for regulating rates, and, therefore, an appraisal for this purpose should disclose the amount actually invested, as in Case 1. This applies to right of way and real estate as well as to other property.

It is true that enhanced value of real estate may increase the amount of taxes assessed against and collected from the railroad company; but, in the usual method of railroad accounting, taxes paid are treated as an item of expense separate from and independent of interest and dividends, and do not, therefore, affect the capital account.

Case 3.—An appraisal made for the purpose of determining the value of railroad property in a contemplated purchase or sale will necessarily differ in many respects from one made for the purposes named in Cases 1 and 2, and the character of the appraisal will vary with the nature and circumstances of the proposed transaction. When the proposed sale is by mutual agreement between parties equally disposed and free to deal, the appraisal is essentially equivalent to the ordinary invoice of industrial or commercial concerns. In other words, its purpose is, chiefly, to disclose market value. Such transactions are mostly between private parties, do not concern the public, and need not be considered here. But where a Government elects to acquire property by the exercise of its power of eminent domain or other coercive processes, regardless of the owner's desire or willingness to sell, the situation is wholly different. If the owner is to be deprived of his property, he is entitled to full and even liberal compensation therefor. This applies, not only to the naked value of the physical property, but to franchise rights and other intangible values, and to any special physical conditions which may make the property especially remunerative. For from any sound business standpoint, present and prospective earning capacity is more truly a measure of value than the cost of the property or its physical value, and is as much a real asset as physical value. Therefore, the guiding principle in an appraisal of the property should be to ascertain its present and prospective productive value, of which original cost or cost of production will be only one element.

The soundness of these views will hardly be questioned in the case of the property of any private person or corporation whose title to the property is clear, equitable, legal, and untainted by fraud. It does not matter when or how such a sound title was acquired, nor whether the property would be of equal, or in fact, of any, value to the purchaser.

In the case of railroads and other quasi-public corporations, created and existing under governmental permission or franchise, however, the title is clouded by stated or implied reservations embodied in the franchises, and this fact must be taken into consideration.

Mr. Whinery. These franchise reservations, it is true, do not confer any rights to ownership or title. The right and power to expropriate the property of private owners without their consent are derived from a different and wholly independent source—the doctrine of eminent domain, which may be exercised in any case for the public good. The rights and powers of Government derived from franchise grants do not usually extend beyond a certain degree of control or regulation for the public good. It is claimed by the legislative, and affirmed by judicial, authority that the Government may intervene in the operation of a quasi-public corporation to the extent of requiring satisfactory service for reasonable charges; but the question, what are reasonable charges, is still an open one. In fact, it is one which is very difficult to decide, not only in a general way, but as to any particular case. It will be conceded by nearly all fair-minded people that, where the service is inadequate and rates are not only oppressive to the public but yield excessive profits to the owners, it is not only the right but the duty of the Government to intervene and to establish relations between the two which will be fair and just, with due regard to the rights of each. It seems to be widely held that reasonable charges are only such as will repay the cost of the service and yield a fair return on the capital actually invested; and fair return is defined to mean little if any more that the prevailing rate of interest. This theory does not recognize intangible property, such as productive value, and, if enforced, might greatly depreciate any value of the property based on its special earning capacity.

The combination, in the hands of the Government, of the franchise right to control, and the eminent domain power to expropriate, property, unless wisely administered, may be a very dangerous one. If a quasi-public enterprise may be forced to extend its service and reduce its rates to such an extent as to cripple its earning capacity and thus to reduce greatly its commercial value, and the Government may, later, seize or condemn the property at the reduced value created by its own acts, the result may be as truly confiscatory as if physical property were taken without compensation to the owner. It is true that the National Constitution prohibits the taking of private property without due compensation; but it is silent on the question as to what constitutes due compensation. This question must be answered ultimately by the Courts. Thus far it has not been answered conclusively and fully as to the intangible values of quasi-public corporations; and until it shall be so answered, the question of a proper appraisal of such properties for the purpose of an enforced sale cannot be decided definitely. There are hopeful indications that the broad ground may be taken and sustained that productive value, in whatever form it may be found, whether physical or intangible, shall be given proper consideration in such appraisals.

In the present unsettled condition of many questions of detail, it is most important that an appraisal of this character should be thoroughly and intelligently considered by the authority ordering it, and that definite instructions as to the principles which shall control it be given to the appraiser, who must then be wholly governed thereby. Mr. Whinery.

Assuming that the owner is entitled to compensation on the basis of a reasonable regard for the productive value or earning capacity of his property, as well as for its physical value, the general principles outlined apply with special force to the appraisal of right of way.

While a country is new and unimproved, a right of way may have been acquired at a very small cost. Not only was the market value of the land very low, but the conditions were such that both temporary and permanent damages were almost negligible; but, to secure an equally good right of way through the same locality after the lapse of 15 or 20 years, when the country has become thickly settled and costly improvements have been made, would be a difficult matter involving great cost. Not only will the market value of the naked land have greatly increased, but numerous and costly improvements will be encountered which will warrant heavy damages. In such a case it would be fair to say that, other things being equal, the old existing right of way is physically worth what a new one in the same vicinity would cost. In addition to this physical value, the existing right of way may possess strategical advantages and facilities for doing business which it would be impossible for the new one to acquire, and, if so, due consideration should be given to the value of these elements.

There are no present indications that either the National or the State Governments seriously contemplate the purchase of the railroads. Nevertheless, it is well to call early attention to the fact that an appraisal for some other purpose, as for regulating rates, cannot justly be used for determining values in expropriation of the properties.

Case 4.—Where the object of an appraisal is to determine the value of railroad property for the purpose of assessing ordinary taxes, the principles and practice appropriate will differ materially from those applying to the other cases considered. This is especially true with regard to right-of-way land. Unless otherwise provided by State statutes, railway land is subject to the same rate of taxation as other lands, and the amount of taxes assessed depends on the valuation placed on the land by the assessors. This assessed value of land may not be, and in fact usually is not, the same as the market value. The ratio between the two values, however, is presumed to be applied to all lands alike. In other words, the assessed value for taxation is rela-

Mr. Whinery. tive rather than actual. An appraisal of right-of-way land for taxation purposes, therefore, should be based on the assessed valuation of similar land adjoining it or in its vicinity. This does not mean, however, that it should be valued at the same unit price as such adjoining land, but rather that its actual value, when determined, should be multiplied by the ratio between the actual and assessed values which may be found to prevail in the locality. In determining value for taxation, the Courts have generally made it plain that neither the use made of, nor the amount of benefit or profit derived from, the land may be considered.

The question to be decided is, therefore: What is the value, other things being equal, of right-of-way land as compared with that of other land in its vicinity?

In seeking a general answer to this question, it may simplify the discussion to use for illustration a hypothetical case which is fairly typical of a majority of right-of-way transactions.

A owns a farm of 160 acres, the fair and reasonable market value of which, as a whole, and of similar land in the vicinity, is adjudged to be $50 per acre. For simplicity, it will be assumed that it is taxed on this full value. Through this farm a new railroad must secure a right of way, 100 ft. wide, requiring, say, 6 acres. The conditions are such that the three elements, fair market value, permanent damages, and temporary damages, are involved. It will be assumed that the parties cannot agree on the amount of compensation to be paid by the railroad, and that the right of way is condemned by a commission of disinterested, competent, and fair-minded men, who exercise their best judgment, after informing themselves of all the facts. They find and report that the market value of the land is $60 per acre, that the permanent damage is $600, and that the temporary damage is $140, making in all, $1 100. The question at once arises, why do they estimate the value of the 6 acres taken, at a higher price per acre than the admitted fair price for the whole farm; and is that action justified? It may be replied, that though it may be true that the severed land is intrinsically no more valuable than that which remains, common business customs sanction the higher valuation. The general analogy to wholesale and retail transactions may be instanced; also the familiar fact that when a tract of land is purchased *en bloc* and divided into smaller tracts or lots, these latter command a higher unit price than was paid for the whole tract. Again, fair market price is said to be that price established when both seller and buyer are equally free and disposed to deal. Now, if for any reason, B wanted to buy a selected part of A's farm, which is intrinsically worth no more than any other part of the farm, and is located so that its severance would not damage the remainder, it

is natural and reasonable that A should ask, and B should be willing to pay, a higher unit price than A would ask for the farm as a whole. It is equally reasonable that the land severed for the right of way should command a higher price than the farm as a whole. What that higher price should be is a matter of judgment, and it must be assumed that the commission exercised its best judgment in the matter. It must be assumed that its award for permanent and temporary damages is also fair and equitable. Mr. Whinery.

The transaction being closed in accordance with the award, the question in due time will arise, how shall the properties be assessed for taxation? Here, the State becomes an interested party, and her rights must be taken into account. The permanent damage of $600 was awarded on the ground that the remainder of A's farm is permanently injured and, therefore, that its value is decreased by that sum. When the tax assessor appears, A may reasonably claim that that sum shall be deducted from what his farm would otherwise be assessed for. The assessor accedes to what seems a just demand, and appraises the remaining 154 acres at $46.10 per acre, and proceeds to appraise the railroad right of way. Here some perplexing questions arise. He may readily decide that the property should be appraised on the value fixed by the Commission, but he discovers that even when this is done, the aggregate assessment on the whole of the property will fall short of the amount the undivided property has been previously appraised at by $540.60. In other words, that on such an appraisal, the State will lose the taxes on that sum. This would obviously be unjust to the State. Further study shows him that in order that the State shall receive the same amount of taxes as before, the permanent damages deducted from A's appraisal must be practically all added to what would otherwise be the appraised value of the right of way. In other words, permanent damages must be considered as accruing to the benefit and value of the severed land, in addition to what would otherwise be its fair market value. He, therefore, lists the right of way at $\$360 + \$600 = \$960$, or at the rate of $160 per acre, and, under these appraisals, the State will receive substantially the same amount of taxes as heretofore. The value per acre thus ascertained is 3½ times that of other land in the vicinity.

The ordinary tax assessor may not enter into such nice reasoning or computation, or reach a like result, but he might well do so, and the soundness of the argument, from the State's point of view, must be conceded.

The conditions described relate to a time closely following the acquirement of the right of way. The question arises whether the conditions then found continue into the future, say, 20 years thereafter. The market value of surrounding land may have increased or decreased

Mr. Whinery. in the meantime, and the naked value of the right-of-way land may have increased or decreased accordingly. The award for permanent damages, however, does not change with time. The injuries to the remaining land, from their very nature, must be regarded as permanent and continuous, and their value must still inhere in the right-of-way land. It is, therefore, still a factor to be reckoned with in appraising the property for taxation purposes.

Coming now to the application of these general principles, it would appear that a proper appraisal of right-of-way land for taxation purposes should be based on at least three elements of value:

First. The fair market value of similar land in the immediate vicinity (excluding, however, adjoining land permanently damaged);

Second. A reasonable percentage to be added for increased value due to selection and severance;

Third. The amount of the permanent damages caused by the taking of the right of way.

If in any given case the records of the original transaction were available and the compensation paid were properly itemized, it would not be difficult to arrive at a proper valuation; but, in practice, especially in the case of the older railroads, these data are hardly ever available, and the appraiser must exercise his best judgment, particularly as to the fair amount of the permanent damages.

In fact, it is found in practice that it is hardly practicable or even possible to consider each individual tract of a right of way thus in detail, and that it is necessary and desirable to formulate some general rule to be applied to the determination of the value of right-of-way land, either as a whole or to certain classes or local sections. This is the more permissible because the right-of-way land of any railroad is likely to be assessed as a whole, and a correctly framed scale of values is likely to give fair average results.

The simplest and, perhaps, on the whole, the best procedure is to assume that the value of the right-of-way land bears some fixed relation to the market value of other lands in its vicinity, and to apply some fixed multiplier of that market value to the right of way. Multipliers varying from one to three have been used in different appraisals, where the purpose of the appraisal has not been clearly defined.

The author states that it is a well-known fact that land for right of way usually costs from 2 to 2½ times as much as the market value of surrounding land. From the writer's experience (not, however, very recent), in well-developed farming country, he would say that these figures are very conservative, and that factors of from 2½ to 3½ would be more nearly correct; perhaps, 2½ would be a low average figure.

It would not be fair, however, to use this factor of cost as a multiplier for determining values for taxation, for it includes the elements of temporary damages and incidental expenses which, obviously, should be excluded from an appraisal for taxation. If we assume that these amount to one-fifth of the total cost, the proper multiplier would be 2, and we would have the general rule that for taxation purposes, right-of-way land should be appraised at double the value of other land in its vicinity. Mr. Whinery.

In the writer's opinion, it will seldom be found that a proper multiplier, determined in accordance with the principles here outlined, will fall below that figure.

It may be, and generally is, claimed by railroad men that the prices thus commonly exacted and paid for right-of-way land are excessive, and that such differences in value are factitious. The fact, however, that they have prevailed for half a century and over the whole country, whether the right of way has been secured by private purchase or by condemnation, would seem to indicate that the general ratio has a substantial foundation in fact.

The author refers to the fact that in the recent re-valuation of the railroads of New Jersey, where the purpose was avowedly to establish a basis for taxation, right-of-way land was appraised by the expert of the State Board of Appraisers at the same unit value as adjoining lands. The writer cannot but believe that if this appraisal shall be finally adopted as the basis for taxing that part of the railroad property, the State will fail to receive the amount of taxes to which it is justly entitled. The land (first-class) was appraised at nearly $47 000 000, but this embraced land for terminals, etc., the separate value of which is not given. Assuming that two-thirds of the whole was properly classifiable as right of way, which should rightly have been appraised at double the value of adjoining land, the State would lose, annually, taxes justly due it on somewhat more than $30 000 000 worth of property.

It is not asserted, of course, that the principles herein advocated will be applicable to every case for which an appraisal is called. Even in the absence of definite instructions calling for a different treatment, a great variety of conditions and circumstances may develop, which may require modified action in individual cases or classes of cases.

The writer's object has been to outline the general principles which, in his opinion, should underlie and control appraisals of this kind of railroad property.

F. Lavis, M. Am. Soc. C. E.—Mr. Wilgus is to be congratulated on the very clear and concise presentation he has made of the basic principles governing the valuation of railroads. He has covered the ground so thoroughly as to leave little opportunity for discussion, and, although the speaker can do little but concur with him, it is, perhaps, not inap- Mr. Lavis.

Mr. Lavis. propriate, in view of the present-day interest in this subject, to express this, and emphasize, if possible, some of the more important points that are made.

In a discussion of the paper* on this subject by Henry Earle Riggs, M. Am. Soc. C. E., the speaker pointed out some reasons why valuations of the railroads might not be an unmixed evil, as seemed to be generally assumed at that time, the conclusion of that discussion being as follows:

"First, that valuations properly made may be the means whereby confidence may be restored, not only in the mind of the general public, but in that of the investor; but, in order to obtain this result, the railroads should urge, with all the power they possess, the necessity of having such valuations made by a body of men, some of whom, at least, should be engineers, big enough to entitle their opinions to the respect of both sides, and thoroughly qualified by training and experience for the work.

"Second, that, as far as possible, regulation should be general or national, so as to avoid the complication of dividing all roads at the State lines, and of having different regulations in different States.

"Third, that there need not necessarily be any relation between rate regulation and rate-making. Rate regulation can well be confined to rates in the aggregate, rate-making applies to the adjustment of individual rates, and must necessarily be the work of men well versed in all the varied elements which control it and the particular conditions affecting the business of each particular road."

The paper by Mr. Riggs was an able discussion of methods of obtaining the cost of reproduction, but the author refused to recognize that the consideration of the so-called "intangible values" had any place in a physical valuation, and he did not consider that the purpose for which an appraisal was to be made should have any influence in determining the value of the property.

It is the discussion of these particular points by Mr. Wilgus which the speaker believes should be emphasized. He points out distinctly that the reason for making the valuation, or the purpose for which it is to be used, has an important bearing on the method of making it, and also states the principles which should guide an experienced man in making a reasonable estimate of at least some of the so-called "intangible values," or perhaps it would be better to say that he has outlined so completely the really tangible values as to have eliminated all or nearly all which can be called intangible, which are included in the unbreakable circle of argument in which it is claimed that the rates are based on values and the values on the rates.

It is shown in the first place how the appraisal of the individual items composing a railroad, without relation to their value as correlated parts of a whole, may be justified for taxation purposes, but is

* *Transactions*, Am. Soc. C. E., Vol. LXXII, p. 174.

not a proper method for determining the value of the property as a living entity, that the "original cost to date" method is untenable on account of the almost universal lack of records of any value and for other reasons, and the conclusion is that the "Cost of Reproduction New" is the only method whereby an appraisal can be made which will be fair and just to both railroads and public, and will give a fair value to the property as a whole and as a railroad, if it be properly made. Mr. Lavis.

Leaving aside the question of taxation, which is really a local issue, the valuation of the railroads is now to be made by the Interstate Commerce Commission principally in order to determine whether the capitalization is fair, whether the rates now being earned give a fair return on that capitalization, or as information on which it may base its judgment in regard to new capital requirements. To one with experience in other countries, where the regulation of the railroads is most jealously guarded by the Government and where it does not act either as a serious deterrent to development or serious inconvenience, this does not appear altogether undesirable or unreasonable, but, whatever one may think about this, it is begging the question to say that no valuation will give adequate values for these purposes. What we have to do is to admit that valuations are to be made practically and solely on this account, and determine how they may be made properly.

The application of any valuation of a railroad property to the adjustment of rates is, of course, complicated, and it can easily be imagined that, on first consideration by any one at all familiar with the intricacies of rate-making, the difficulties appear insurmountable. Suppose fair valuations are arrived at; suppose a fair return on this value is fixed, and the aggregate of the rates raised or lowered by a certain percentage to give this assumed fair return; how, it is asked, are the differences in management and the thousand and one other inherent differences in physical characteristics and conditions to be overcome? There would be no competent answer to this were each railroad to be taken as a unit or an attempt made to regulate individual rates, but, inasmuch as it would be difficult to change a rate on, say, the Pennsylvania between New York and Chicago without making a similar change on the Baltimore and Ohio, Erie, New York Central, etc., it seems impossible to use such valuations as may be arrived at for any other purpose than the regulation of the rates as a whole over a distinct section. That such a method of rate regulation is a practical one has already been recognized by the Committee of Presidents of the Eastern Railroads. For some time that Committee has endeavored to obtain permission to raise certain individual rates which were admittedly too low, but the opposition from the shippers has prevented favorable action by the Interstate Commerce Commission.

Mr. Lavis. The Committee, therefore, has now decided to ask for a blanket increase of a certain percentage on all rates throughout the Eastern territory, as there seems to be far less objection to the rates, *per se*, than to any change in the relation of certain individual rates to certain others, and this is quite in line with the speaker's conclusion in his discussion of Mr. Riggs' paper.

The trend of practical endeavor, therefore, as well as a common-sense view, seems to lead to the conclusion that the only adjustment of rates can be a blanket one, covering a large section of the country where prevailing conditions are generally similar, and to the speaker this seems to put the question of valuation on a basis where many of the objections, which may be and are quite properly raised, against valuation of separate roads or parts of them, can be overcome.

Most of the arguments against any theory of basing rates on valuation are founded on the inherent differences in physical characteristics, methods of management, local conditions, etc. In an address delivered before the Southern Commercial Congress, in April, 1911, by Charles Hansel, M. Am. Soc. C. E., it was shown that a higher or lower rate of gradient on two or more competing lines might materially affect their value. He assumed three lines, each 100 miles in length, the first with 0.3% grades, the second with 1.0% grades, and the third with 1.136% grades, and showed that for a tonnage of 10 000 per annum the capitalized value of the additional cost of hauling the tonnage on the two latter might amount to some eight or ten millions of dollars.

Of course, the assumption of two roads between the same termini with essentially different physical characteristics is the *reductio ad absurdum* argument. As a matter of fact, when there are two or more roads between the same termini, or which reach the same points, the rate between these points for the same class of commodity is the same, no matter what the operating conditions of the properties are. If the valuation of the property is for the purpose of purchase or sale, the operating value, as affected by the physical conditions or physical characteristics, should be considered, but, if the valuation is used as a basis for fixing rates between competitive points, it cannot be.

Such a case as that cited above is, from a practical standpoint, purely theoretical, and it may be admitted at once that, if there were any two such roads, no valuation would be a fair basis for fixing the rates on them. If the rates were fair for one, they would be likely to be unfair for the other; looking at the matter broadly, however, there can be no more reason in the valuations to be made by the Interstate Commerce Commission for comparing physical conditions or physical characteristics of competing lines than there is now. No two lines could be more dissimilar than the Pennsylvania and the New York Central between New York and Chicago, and yet the rates are

the same. If the Erie Railroad spends a large sum of money and makes improvements which will reduce its ruling grade to, say, 0.3%, it can hardly be expected that on that account it will be compelled to adopt a lower rate than it charged before, because it can operate more cheaply, or charge a higher one because it has spent a lot of money. Mr Lavis.

There is, perhaps, a certain class of individual rates between local points which are, and probably will continue to be, subject to investigation by the Interstate Commerce Commission or the State Boards. The regulation of these rates, if necessary, must inevitably be based very largely on the cost of service, and, as a basis for determining this, or at least as a starting point, there would seem to be nothing better than an adequate appreciation of the value of the property involved. It seems improbable though that there could ever be considered even, the proposition to fix rates separately on, for example, the Pennsylvania, the Baltimore and Ohio, the Erie, and the New York Central on the basis of the valuation of the individual properties, or even a combination of the values of the physical properties with the capitalization and earnings of each individual property; but that need not prevent the use of a fair valuation taken together with an appreciation of the cost of operation, as a basis for a general advance of rates which might be shown to be necessary in order to provide a fair return to the stockholders and an inducement for the provision of necessary new capital.

It seems to the speaker to be entirely outside the realms of practical discussion to consider for a moment the actual costs of any of the parts which go to make up the railroad as a whole, except as such costs might be a sidelight on their value to-day, or we would never get anywhere. Take, for instance, right of way: some of it may have been given free; it may have been paid for at ten times its value; it may have been obtained by right of eminent domain and the legal expenses charged to something else. Rails were bought at $20 or $40 a ton which would now cost $30. A tunnel was built at a cost to the railroad by which the contractor lost money; another at a cost which allowed a good profit. The whole road may have been bought at a foreclosure sale and so on, almost without end.

The speaker was asked some little time ago to prepare a basis for the determination of unit prices to be used in connection with the valuation of a certain railroad. In looking up actual costs of rock excavation to this road, many contracts were examined dating from 1880 to the present time, the prices ranged from 65 cents to $2.50 per cu. yd., and all within a comparatively narrow range of territory. This, of course, is nothing new, but surely no one with any experience could expect to base the value of the rock excavation on, say, the

Mr. Lavis. Erie Railroad as it exists to-day, on such original costs of parts of it as might be found by an examination of the records. There are altogether too many complications to make it possible to consider original costs, except such as those in which all the governing factors are known which may be used as a guide in determining present-day values. A rock cut may have been taken out originally at $1 per cu. yd., and a slice taken off the side for another track might easily have cost $2, or even under certain conditions less than it did in the first place. It seems to the speaker that, looking at the matter from a practical standpoint, these and innumerable other complications of a similar nature cannot be considered now. We can only consider what it would cost to take that cut out if we had to do it to-day and get it in the shape it now is, and that is quite within the range of the ability of practical men. Really, the cost of reproduction new is the only logical, practical way that will give results which, at least, will be as fair to one road as to another, and as fair to the public as to the railroads.

In many countries where the Government has retained full control over transportation enterprises (and, after all, if at all fairly exercised, this is far better than either unbridled private enterprise or Government ownership), the amount of the capitalization which will be recognized by the Government is definitely fixed. The first cost is determined and approved during and on the completion of the construction, after a satisfactory investigation of the books, and additions made yearly to cover the additional capital expenditures. In the United States, for many good and sufficient reasons, and others equally bad and insufficient, we have let the railroads alone, and now the Government is trying to regain the control it should never have relinquished. As long as this control is fairly exercised, we shall be all right. To exercise it fairly, those charged with it must have knowledge, and one of the things they should know is the value of the property they are controlling.

This value can be obtained fairly, but, it seems to the speaker, only by putting everything on a common basis. There can be no fooling with such questions as "right of way has no value because it has been given to the railroad," or that it "has not increased in value," or that "if we wanted to reproduce it to-day we would not pay a farmer three or four times the acreage value if we cut through between his homestead and the rest of his land." Such speculations as these open up vast vistas of discussion to which there is no end. The "Cost of Reproduction New" is entirely practical and one well within the range of the understanding of any fairly intelligent person. To obtain it fairly needs practical experience, training, and a knowledge of railroad conditions, and Mr. Wilgus has done a service in calling atten-

tion to some of the matters which, if the work is to be done properly, must receive most careful consideration from those who he truly says are "burdened with a Herculean task" in undertaking to value all the railroads of the country. Mr. Lavis.

The necessity for the employment of competent men is pointed out, and this fact should be emphasized, for though it may be generally admitted that a certain number of competent engineers are required, it is not always conceded that railroad as well as engineering experience is required in many of the comparatively minor, or at least intermediate, positions.

Even the first step, the compilation of an accurate inventory of the property of a railroad, must be taken by men in the field who have become capable by long association with railroads in the making and in their subsequent maintenance. Take, for instance, the one item of the earthwork on a railroad, the questions of settlement of embankments built across marshes, the settlement of the embankments themselves, the estimation of slides, the classification of the material, haul, the general seasoning of the whole roadbed, etc., cannot be properly answered or values estimated by any but men of experience who can and do judge the conditions on the ground. The task of making an inventory and fixing prices of the measurable items, although one of great importance, is well within the range of an organization which can be developed from the large number of competent engineers available who have had fairly long and wide experience in railroad construction.

The really important matters pointed out by Mr. Wilgus, however, are the estimation of the value of the two stages of the development of a railroad preceding and succeeding the actual construction period, and the speaker believes, with him, that an equitable valuation of the costs to a railroad during these two periods is not an impossible or impractical task, and, if fairly made by men of widest experience in railroad affairs—not merely engineers with only construction experience—will largely cover the so-called "intangible values."

Until very recently the speaker thought that the position taken by Mr. Wilgus, and as expounded in this paper, was so obvious that there could be little chance for argument, but there still appears to be a large number of otherwise intelligent people who continue to think that the valuation of railroads is necessarily different from the valuation of any other business property, in other words, that the valuations to be made are to be simply an inventory of ties, rails, locomotives, etc., with a price for each, and that the "good will," etc., is to be ignored. We have, perhaps, rather ignored the discussion of the "intangible values," because we have not been able to see quite clearly how we were going to estimate their value, but that may be due to the same state of mind we are often in, when approaching any

Mr. Lavis. problem of importance which may come up. We may have a bridge, or tunnel, or anything else, to design and construct. When we start we may not know exactly how it is going to turn out, even though we know we can do it; we know there are difficulties; we know that the first tentative designs will be all changed; but we also know that if we keep at it long enough and with a sufficient degree of intelligence, backed by experience, we shall work it out. It seems to the speaker to be the same way about valuation. Proper consideration will show that many of the so-called intangible values have considerable substance, and a fair determination of their worth can be arrived at by a proper process of intelligent application. Mr. Wilgus has drawn a broad outline of the matters to be considered; it is for those who have the work to do to fill in the details.

There might be endless discussion, of course, in regard to what these values should be, but it looks as if we had come to the point where some one will have to decide. It is not a question of absolute right, but of coming to a decision which will be as fair as possible to all concerned. There is nothing impractical in this, the absolute right of the majority of disputes is seldom reached, most decisions are compromises or, at least, the best judgment of some man or body of men, and it is usually only necessary that we be assured of the competence and impartiality of the tribunal to permit us to acquiesce in the decision. It is necessary, however, that we be assured of the competence as well as the impartiality, and that all pertinent matters will receive due consideration. The necessity of this latter should perhaps be emphasized most particularly in connection with the estimation of the costs or value of those items which exist but are not easily or physically measurable.

Mr. Churchill's remarks seem to convey the inference that he prefers the method of "original costs", and that he believes this method to be practical.

As the writer understands the situation, the ruling of the Courts and of the Act creating the Interstate Commerce Commission makes it necessary, or at least desirable, that original costs shall be shown wherever possible, but only as a guide to the value of the estimate showing the "present fair value."

The *Railway Age Gazette** seems to express the prevailing opinion in regard to the present fair value quite clearly, as follows:

"When property is taken under the power of eminent domain, the value on which the compensation paid for it must ordinarily be based is its value at the time it is taken. The amount which it has cost its owner may be considered in the endeavor to ascertain its present value; but it is the payment of its present value, and not of what it has cost its possessor, which constitutes just compensation. On the

* September 12th, 1913.

same principle the valuation of a public utility to establish a basis for the regulation of its rates must, it would seem, be based on the present value of its property. That this is the correct legal view certainly seems to be a just inference from language repeatedly used by the federal courts." Mr. Lavis.

If the original contracts, agreements, or book costs, can be found, showing exactly the cost to a railroad of a certain part of its property, it is, of course, very good evidence tending to prove the value. The difficulty of applying these values, however, and one which it hardly seems necessary to point out, is in identifying the structures covered by the original costs with those existing to-day. Is there a single cutting or embankment on any railroad anywhere in the same condition it was 5 years ago? to say nothing of those on which the original work was done 75 years ago. (There were nearly 7 000 miles of railroad in operation in 1850, mostly in what is now official classification territory.) Even if we had exact records of the original cost of each cut and fill and of all the work done on it since, which should properly be charged to capital account, would that give a "fair value" for the property in use at the present time, and on which the regulation of the rates and of the amount of new capital requirements are to be based?

The fact of which we, as engineers, are all very proud, and which is undoubtedly true, as Mr. Churchill says, that most, if not all, contracts for railroad construction are honestly and fairly made and executed, does not seem to the writer to be any necessary criterion that they represent a fair value of the property in use to-day.

There is also another objection to their use, that, from the standpoint of the public, which rightly or wrongly looks on the railroad as an opponent, these contracts will be looked on as *ex-parte* evidence. As evidence tending to prove the correctness of the inventory, for showing exact quantities, classification, character of materials, etc., every jot of evidence shown by the old contracts and plans should be sifted out and used for what it is worth, and this is one of the smaller tasks in making a correct inventory, which, in itself, might be called stupendous, except that we have to be somewhat chary of such adjectives nowadays when nothing seems too large to be undertaken.

It is entirely feasible and a matter of common practice to base a fair present-day value on the consensus of opinion of competent men engaged in practical work of the nature under consideration, and whose experience entitles their opinion to weight. Any other value, say that of 5 years ago, or 10 year hence, or even the actual price paid, is open to controversy as not being fair. If the value of 5 years ago is taken, why not take it for 10, 20, or 50 years ago? If a railroad is to be criticized, as it may be, for awarding contracts for

Mr. Lavis. supplies at exorbitant prices to friends of officials, even if its construction contracts are fair and just—and who shall say they always are—why is it not also entitled to credit when, for any reason, it gets supplies or construction done for less than the present-day market value, as it sometimes does? All these matters, however, invite controversy, but a fair valuation, made by competent men, based on the present "cost of reproduction new" is practical, can be defended, and, if properly made, will be accepted. It seems to the writer doubtful whether any other value can or will be.

The point brought out by Mr. Churchill, that the records of all variations shall be kept in the forms of accounts prepared by the Interstate Commerce Commission, would seem almost to bar the use of original records, except for their worth as evidence tending to show the present fair value.

Mr. Crehore. WILLIAM W. CREHORE, M. AM. SOC. C. E.—This admirable paper shows the author to be thoroughly familiar with his subject, which he treats in a comprehensive and exhaustive manner not to be expected from any other than a railroad official of wide experience. On some of the special points, however, which are matters of opinion only, a layman may be permitted, perhaps, to differ from the author's conclusions without exposing himself to the charge of presumptuousness. On this basis, the speaker wishes to take one exception to the opinions advanced in the paper.

In enunciating the basic principles of procedure and the several ways of determining the physical value, Mr. Wilgus refers to the method known as "Cost of Reproduction, Less Depreciation," and states that, for rate-making purposes, it seems to him that the physical depreciation should not be deducted from the cost of reproduction new. His reasons for this are "that all expenditures for renewals and repairs should be charged to working expenses, and not to capital," and that "normal depreciation is not to be treated as a wastage of capital, but as an element in the cost of operation that is covered by the rate." He then goes on to state that the law requiring "that valuations shall be revised from time to time as improvements and other changes are made, in effect prohibits the deduction of depreciation, as otherwise the constant addition of amounts expended in the replacement or renewal of items that had been included in the valuation at their depreciated price, would be a violation of the rule of the Interstate Commerce Commission that operating expenses should not be charged to capital."

The fact should not be lost sight of that betterments and improvements, when paid for out of income, are just as much property as if paid for out of capital, and, as assets of the company, they should be included in the inventories made from time to time. It is the evident

intention of the Interstate Commerce Commission to draw a definite and distinct line between funds used for extension and enhancement of railroad property, and those used for the prevention of undue depreciation of existing property; in other words the distinction is between repairs and renewals necessary to maintain the original investment at its full efficiency, on the one hand, and such betterments and improvements, on the other hand, as constitute permanent additions to the original investment. In order to protect the stockholder, the property paid for by his original investment should be kept up by repairs and renewals as nearly as practicable to 100% efficiency, or a fund should be established to reimburse him for the difference. Mr. Crehore.

It is customary to look at depreciation as a stockholder's liability, but, in truth, it is his liability only because he has received in dividends at some previous period the money which should have been expended to keep the property up to 100% efficiency, or which should have been put into a fund to be used for that purpose. It is unquestionably the moral duty of the management to keep the company's property up to its full efficiency, or to establish out of the income an equivalent reserve fund. When this is done, there can be no liability for depreciation on the part of the stockholder. Whenever this money is paid to him as dividends, however, instead of being put into the property, he is then actually receiving back part of his original investment, on the instalment plan, as it were. If, then, the stockholder has received this part of his investment back, it certainly remains no longer among the assets of the corporation; hence, to obtain a proper value of the assets, for rate-making purposes, or for any other purpose, the amount of this depreciation should be deducted. The opponents of this view certainly will not contend that rates should be made proportionate to assets which do not exist in the corporation.

The author's view that depreciation should not be deducted is based on the stockholder's liability to be called on at some future time for fresh capital to put the property back to 100% efficiency. Until that time comes, however, and the stockholder does thus put his money back into the property, is there any reason why the rates should be large enough to pay him dividends on that portion of his investment which he withdrew from the corporation, as well as on that which he still has in it? Or if, as is usually done, the property is put back to full efficiency by the use of future income (that is, through an issue of bonds), is there any good reason why the stockholder should be the sole beneficiary from that operation which has cost him nothing? Does it, in the first place, seem just to make the public pay rates high enough to cover dividends on the depreciation of the property, when the equivalent of that depreciation has already been paid back to the stockholder? In the second place, is it just and right that the rates should be high enough to cover dividends on the original value of the property in-

Mr. Crehore. cluding depreciation, and, at the same time, high enough to retire an issue of bonds necessitated only because of the withdrawal of the deprecation reserve fund by the stockholders?

To make the rates high enough to cover future betterments and improvements and to lay the money aside for this purpose is one thing; but to make such rates and then devote the money thus raised to dividends based on the original value of depreciated property cannot be justified. The stockholders should not expect to "have their cake and eat it too." To argue that this is customary does not justify it as good practice, nor does it appeal to the fair-minded gentlemen composing the Interstate Commerce Commission as being either good theory or good practice. Any corporation which is handled so as to allow its property to depreciate without providing a fund out of the income as the equivalent of this depreciation, cannot claim that the full investment of the stockholders is represented in the assets of the corporation, and, therefore, cannot expect to be allowed a rate based on fictitious property just because it has been capitalized.

The market value of any property other than a public utility, that is, any property unregulated by law, is a direct function of its income, fluctuating with the income. This market value apparently is unrelated either to the intrinsic value of the company's assets or to the amount of capital invested. Investors look only at the rate of income and the safety of the investment. If, for instance, the stock of a reliable concern pays 40%, this stock cannot be bought at par; but the market price will be well up to a point where dividends on it will be normal.

Now the difference between this market value of the "going" concern and the actual intrinsic value of its real assets is ascribed to the "good-will" of the business, or the "franchise" value, as it may be called in the case of a public utility, meaning thereby the value of its intangible assets. If the business is highly prosperous, the good-will or franchise value is correspondingly great. The actual intrinsic value of a going concern's real assets does not change merely because of the advent of prosperity or the reverse, but the good-will or franchise value does vary for this cause. With some very highly developed monopolies, the good-will value of the business is the largest asset, the intrinsic value of its property being merely incidental.

This being so, it is evident that the problem before the Interstate Commerce Commission, in regard to railroads, is to reverse the natural operation of an unrestricted monopoly. Instead of "charging all the traffic will bear" and determining the good-will value of the business by the amount of income, as is common with a private corporation, the Commission's problem is to ascertain the value of the assets (intangible as well as tangible), and then to make the income conform with it by virtue of the rate-making power. This difference be-

tween a public utility and a private corporation seems generally to be unrecognized. In the working out of this problem there is very little to be learned from the example of private monopolies, excepting as they may be said to teach "how not to do it". In the Commission's problem it is necessary to prevent all exaggeration in respect to the good-will or franchise value in determining the value of a railroad's assets. Mr. Crehore.

A railroad is a monopoly created by the law-making power primarily for public uses. A private corporation, on the other hand, is created primarily to make money for its stockholders. A railroad's charter is granted by a special agreement with the Legislature, and gives it an exclusive monopoly in its own territory, except at and near the terminals. Such a monopoly must of necessity be regulated by some common authority, in order that the communities through which it operates shall not be discriminated against in comparison with those similarly situated on other railroads, and for the further reason that the railroads are practically indispensable to all business interests. Each year sees the business of the country more and more dependent on the railroads for the interchange of commodities.

For these reasons, also, it is necessary that the public authority in control of the railroads should endeavor to have the rates fixed as low as is consistent with the value of the capitalist's investment. This investment, however, certainly does not comprise betterments and improvements which have been paid for exclusively out of rates (which, for this reason, are excessive rates) charged to the public. In this respect a railroad's opportunities to make money for its stockholders are far greater than those of a private corporation competing for a livelihood; and railroad investors have so long been accustomed to the advantage they possess for making the public pay for anything they want, that it will not be an easy matter to attract them when these opportunities are removed, although the return on their actual investment should still remain as good as, or better than, that of corporations in private business.

Neglect to provide depreciation funds is in the same category, both financially and economically, with the omission to establish sinking funds for outstanding bond issues. Both practices have brought about an enormous inflation of values by the introduction of fictitious capitalization. Bonds have been left outstanding long after the property or equipment purchased with their proceeds has become useless; or, if refunded, the effect is the same, because stockholders claim their share of the earnings beyond the lifetime of the property to produce which their proceeds were applied. Whenever an issue of bonds is refunded, which ought to have been paid off by a provision out of the earnings, its claim is added to that of the new bond issue for a

Mr. Crebore. share of the future earnings. Such bonds as are not co-terminous with the life of the property originally purchased with their proceeds, serve to increase the burden of inflation, because the security that was once behind them is abandoned, useless, or obsolēte. To provide income for bonds like these is to carry a fictitious investment on the books, and with every refunding of debts that ought to be retired, the load of over-capitalization receives a new increment.

It has come to be the popular conception that to capitalize anything is to provide an income for it. But capital is wealth; therefore, when an income is provided for something which does not exist, there results a form of capitalization which adds no wealth to the property at all, but is fictitious capitalization or over-capitalization. It provides an income for fictitious investment. Property which has become obsolete or superseded by other property ceases to have any earning power, and is no longer entitled to any income. Hence the bonds, from the proceeds of which it was originally purchased, must be retired by the earnings of the period covered by the life of the property in order to prevent over-capitalization or inflation.

An exact compliance with this theory is, of course, not possible in practice. With the railroads of the United States, progress and development have caused such unforeseen changes in recent years that it has not always been possible to arrange bond issues which should terminate with the life of the property represented by them; but there could have been, and should now be, made a more vigorous effort to retire such issues as have outlived their usefulness, by payments out of the earnings rather than by refunding, and it must not be expected that the public will permit rates to be increased in order to enable the railroads to make a readjustment which has been necessitated by excessive rewards to stockholders. The stockholders have "eaten their cake" and that of the bondholders too, and their efforts to get the public to feed the bondholders no longer deceive the majority of those interested.

The great incentive behind this movement for the valuation of existing railroad property evidently lies in a desire to have more complete and definite knowledge as to the size of this load of over-capitalization carried by the railroads. The extent to which progress and development have contributed item after item to the load of over-capitalization in recent years is not realized by many; and the inflation itself is totally denied by others. Naturally, there will be some among the latter who will be strongly impelled to complicate the valuation work, and to seek out every possible pretext for making the assets, both tangible and intangible, look as big as possible. It should not be inferred from this statement that Mr. Wilgus is credited with any but perfectly sincere and honorable motives in his elucidation of the subject.

Throughout the paper he has repeatedly stated that the method of obtaining the physical valuation should be chosen with reference to the purpose for which the valuation is made; that is, for purposes of rate-making he would recommend a method giving a different result from that obtained by the method he would recommend for the purpose of selling the property. Perhaps still another method might be recommended for purposes of taxation. Much has been said recently about the increasing burden of taxation on railroad properties, and there is no doubt a tendency on the part of local authorities all over the country to assess railroad property more nearly at its true value than has been customary in years gone by. Mr. Crehore.

Without in any way begging the question of physical valuation, the speaker takes the ground that the easiest and simplest way out of a complex and difficult situation, and one that would wipe out an immense amount of inflation, would be to order each railroad to put its own value on its property of all kinds, and to have it understood that the same value would be used in assessment for taxation as well as for acquisition of the property or for any other purpose. In other words, if there was any inclination to place a low valuation on the property for purposes of taxation, this would be counteracted by the knowledge that the Government would use that set valuation for rate-making purposes and for acquisition if desirable. With the aggregate of all these self-imposed railroad values thus put before it, the Interstate Commerce Commission could then determine fair and equitable rates based on these valuations. At least, if the rates thus determined were not fair and equitable, the railroad companies could not complain of the valuations on which they were based.

Our whole system of direct taxation is cumbersome, discriminatory, and unjust, putting a premium on shiftlessness. The man who keeps his property in good repair, his buildings and fences in order and well painted, is assessed far above his neighbor whose acreage is the same, but whose buildings and fences are dilapidated and broken down. If the estimate of each was accepted as the value of his own property, and if such valuation was registered in the public records to be used for all purposes, including taxation, the authorities would be relieved of a vast amount of burdensome detail, and if any injustice were done, the victim of it would have only himself to blame.

It is a poor rule that does not work both ways. What a boon such a system would be to the railroads for purposes of acquiring rights of way for extensions! Condemnation proceedings could be avoided, untold agents' commissions could be left out of consideration, and, best of all, an exact estimate of the cost of any proposed right of way could be obtained in advance merely by consulting the registered values of the property which it was desired to condemn. If the law

Mr. Crehore. authorized the condemnation, then the owner of the property would be obliged also by law to sell at his own registered figure. If property owners were permitted to change their valuations as often as once a year on a date fixed by law, and, when they failed to alter them, if the last previous valuation should be held to be valid until a new one was submitted on the regular date fixed for the purpose, speculation would be beneficially regulated rather than checked, and to the extent of the variation in taxation the State would share in the speculation, which is as it should be.

Under a general system of this kind applicable to all corporations and individuals alike, the unearned increment on unproductive but well-located property would be automatically and largely diminished. In any event, the full import of this feature would be shared by the State, and any tendency to exaggerate it would be checked by the proportionate rise in taxes. The author very justly complains of the excessive prices usually paid by railroad companies for property to be acquired for extensions and betterments; but, under the system just described, railroads would have to pay for such property no more than anybody else. Thus a prevalent cause of over-capitalizing railroad property would be removed, and not only would there result a relief in the financial burden, but a lower basis for rate-making purposes.

It is because of the very eminent fairness of this method of valuation that it is not likely to be popular. Property owners like to collect damages from some one when under compulsion to sell what is their own at a fair price for the public benefit. In fact, when a railroad is the purchaser, they too often look on the transaction as more beneficial to the railroad than to the public. At least they know that the railroad company is more likely to bid high for their property than any other individual purchaser, because land is capital, and the railroads are paying very high premiums for capital in any form. The intense sentimentality over property rights so prevalent in the United States would be sadly shattered if a man's rights to his property were reduced to just what he himself made them, and was able to pay for. As it is now, a man's property rights (within certain limits) are anything he chooses to think they are, and he does not have to sustain his exorbitant rating of them by one single item of expense beyond that of his neighbor who is similarly situated, but whose ideas are more moderate. Property owners in this country to-day are a highly privileged class, but are placed on a most unjust and discriminatory footing with respect to each other on account of the present system of direct taxation which is without uniformity, unsupported by reason, easily responsive to corrupt influences, and, most of all, is unstable or unreliable for revenue-producing purposes. The burden falls most heavily on those who refuse to take dishonest advantage of the system's weak points.

The important question in rate-making seems to be, not what is the intrinsic value of the property, but rather what is the capitalist's actual investment, and what portion of the earnings should be allowed to be capitalized and to become a basis for further dividends to be met out of the rates. It will be admitted that the rates must cover, in addition to all administrative and operating costs, (1) repairs and renewals necessary to keep the property up to its full efficiency, or the equivalent in the form of a fund for the protection of the stockholders, and (2) a reasonable return on the capital actually invested. To increase the rates beyond this in order to provide betterments and extensions is no different whatever from paying the stockholders larger dividends to enable them to reinvest immediately in the business, because, eventually, these betterments and extensions are capitalized and handed to the stockholders as a present. Of course, money for betterments and extension does come from previous dividends, either of these or of other stockholders, in this or in other business; but the question is, what dividend rate ought to be considered sufficient to enable the stockholders to reinvest the needful amount in betterments and extensions and have something left for themselves. With a private corporation the answer is, make as much money as you can; but with a public utility the object is to serve the public just as cheaply as possible, and, therefore, the problem begins with a determination of the minimum return which can be offered to attract capital. Mr. Crehore.

It has been said by railroad financiers testifying under oath that without the practice of capitalizing earnings and offering stock bonuses with bond issues, capital could not be attracted in sufficient quantities to take care of the necessary improvements in railroad operation. These statements are doubtless true, and indicate the extreme competition for capital at present prevailing in the United States. It would seem that capital ought to know its own interests well enough not to demand such large rewards that the business itself must eventually be throttled in order to satisfy its cupidity. This situation results from an interference with the natural law of supply and demand. As the wealth of the country gradually becomes concentrated in the hands of fewer individuals, the amount of capital outside of their control becomes less. All capital is wealth of some sort. As these people absorb an increasing proportion of the total wealth, they acquire control of a correspondingly increasing proportion of the total capital, and automatically with the growth of their monopoly the available sources of capital diminish. Under such competition for capital as results from this monopolistic control, it cannot be expected that railroad rates will be kept low if all the demands heretofore mentioned have to be supplied from the earnings.

The President of the Pennsylvania Railroad Company issued a statement on May 12th, 1913, in which he said:

Mr. Crehore.

"In England the policy of railroad companies has been to pay out currently to stockholders nearly all of the net earnings, and provide for all improvements out of the proceeds of sales of capital stock. If the investors in the stock and bonds of the Pennsylvania Railroad had supplied directly all the money which has been invested in the transportation property of this Company, and if they received the entire annual net earnings from the operations of such property, they would today be getting only 4.83 per cent. upon their actual cash outlay."

Two conclusions may be drawn from this statement: one is, that if capital can be obtained on such a basis in England, it can be so obtained in this country from England by removing from our properties that most objectionable feature, over-capitalization or inflation; the other is, that if the capital for the betterments above referred to did not come from the Pennsylvania Railroad stockholders it must necessarily have been collected out of the public, or (if through bond issues) is now being collected out of the public. In other words, the difference between our method of financing railroad extensions and England's method is, that over there the capitalist puts up all the capital and receives a fair return for it, and here the public puts up most of the capital and makes a present of it to the capitalist, not only getting nothing in return for it except the improved service, but also paying rates large enough to cover dividends on the gift as well as on the capitalist's actual investment. If prevailing methods are persisted in, the logic of the situation will force an answer to this question: in financing a railroad, if the rate-payers are to furnish the lion's share of the capital anyway, what is the use of having any stockholders at all? They are merely an incumbrance.

About such a state of affairs comment is superfluous; a complete change in the system is called for. The reform has been well inaugurated by the authoritative movement to obtain a careful and complete physical valuation of railroad properties, and this movement should have the earnest and candid support of all who are interested to see better times for the railroads as well as for the communities they serve. With the knowledge thus gained, the next step will be to eliminate the load of over-capitalization now carried and to begin retiring bond issues out of the earnings, instead of refunding them and allowing their share of the earnings to pass to the stockholders. The class which has so long been getting something for nothing must be taught by the removal of such opportunities to be satisfied with a fair return on its investment.

To correct a misunderstanding which seems to have arisen in regard to the speaker's proposed system of fixing property values for taxation and other purposes, it should be said that by this system there would be no prohibition against any individual placing his valuation figure as high as he pleased. It has been suggested in this discussion that

it would be a great injustice to a farmer to take a strip of his land for railroad purposes at the same rate of compensation as he would ask for the whole property. Of course it would. A strip dividing his property for any purpose, whether for a railroad or not, would damage what remained to him to a certain extent, but there would be nothing to prevent him from putting a valuation high enough to satisfy himself on that portion of his property through which it was at all likely that a railroad would pass, neither is there any reason why he should not place a higher value on some parts of his contiguous possessions than on others, and have it so registered. The only requirement imposed on the farmer by the system would be that he should pay taxes according to his own valuation. Mr. Crehore.

This would not be the hardship that some seem to think. There would be no general increase of taxes by an adoption of this method; the total amount collected for public uses will not vary because of a readjustment of values every year. The total amount of levy on a community remaining the same, the readjustment of values would mean merely a redistribution of the tax levy to correspond with the readjustment, which is as it should be. If a man thinks that his property is worth more than his neighbor's, he should be willing and ready to sustain his rating by paying proportionately greater taxes. The speaker cannot conceive anything unfair in such a method of taxation. The injustice to a community comes from announcing and maintaining values which are fictitious and misleading, or at variance with each other—low taxable values on the one hand and high selling values on the other. The merchant who asks one price from one customer and another price from another has a poor standing in any community. A sells property to B at a certain figure which both agree to keep secret. The newspapers, in the interests of speculators in the neighborhood, announce the sale at a figure very much higher than—perhaps twice as high as—the actual payment made by B, thereby misleading all property owners in the vicinity as to the real value of their own investments, and scaring off prospective purchasers or lessors who might have been attracted by a knowledge of the correct values prevailing in that community.

Discrimination in some form or other is at the bottom of all our industrial and financial troubles in these days. The injustice of our present system of taxation lies in recognizing one value for taxation purposes and a different value for purchases, and in discriminating between property holders who are similarly situated. Such a system promotes underhand work, and the tendency is to throw the burden of taxation on those who are not so well equipped as others to battle for their rights. If those who adhere in blind devotion to the present system would take into consideration the confusion it makes, the

Mr. Crehore. wrongs it perpetrates, the corruption it invites, and the false swearing it occasions, there would be a different feeling in regard to making a change.

As to what proportion of the betterments and extensions to railroad property should be provided from the proceeds of the sales of capital stock, and what part should be paid for out of the earnings—in other words, as to what extent surpluses should be capitalized—it should be understood that the speaker's whole contention applies to public service corporations regulated by Governmental authority, and not to Government-owned enterprises. Of course, where there are no stockholders, as in the latter case, it is proper that the rate-payers should furnish all the capital needed, as they are the sole beneficiaries of the increased earning capacity due to the betterments. The rate-payers, then, being the owners, whatever is done with the property redounds entirely to their injury or benefit as the case may be.

In the beginning it was assumed by the writer (and this assumption has been confirmed by the author) that the subject of the paper was based on the question: What is the proper way to ascertain the true physical value of railroad property; and only incidentally included the question: Shall this intrinsic value be used for the purpose of rate-making or not?

The writer's stand on the first of these questions was and is that a proper physical value of the assets can only be obtained by finding the cost to reproduce at prevailing prices and deducting a proper amount for depreciation, and that this resulting valuation should be used for all purposes where the correct physical valuation is called for. When, however, the correct physical valuation has been found, there still remains the question whether rates should be based on that or on something else. These appear to be two separate and distinct questions, although several of the parties to this discussion have treated them as if they were interdependent.

The utter hopelessness of getting a fair physical valuation by the Original Cost Method and the stupendous amount of work involved in getting it by the Cost to Reproduce less Depreciation Method was so impressive as to cause the writer to suggest the alternative of letting the railroads put their own valuation on property of all kinds. And yet—whereas an accurate value of the property could not possibly be ascertained by the Original Cost Method—there would be no difficulty in showing from the books just what money each investor had paid into the treasury; and, in the case of a public utility property, it is contended that this, taken together with a reasonable portion (not all) of the value of betterments and extensions, should be the basis of the investor's return paid to him out of the rates, the public being the beneficiary as to all the remainder.

Mr. Humphreys.

ALEXANDER C. HUMPHREYS, M. AM. SOC. C. E. (by letter).—The writer is in complete agreement with Mr. Wilgus' statement that the task of valuing the railroads of the United States is Herculean. He has seen nothing to lead him to think that our politicians have any adequate conception of the magnitude and the almost, if not quite, insurmountable obstacles in the way of a competent solution of all the questions involved.

There have been some expressions from the Commission charged with the chief responsibility, to indicate that even its members do not fully appreciate the grave responsibility which rests on them.

In addition to the inherent difficulties, one has to face what the writer believes is a too common fault in the United States to-day, fear —on the part of those directly interested—of criticism and denunciation from an uninformed and prejudiced public.

Already men have been appointed to positions of influence in connection with this work who have demonstrated that they are incompetent, and, what is worse, are willing and able to labor for the destruction of legitimate property rights.

This and much more that might be said indicates that we have not to-day in the United States a proper background for these appraisals. There is need of an effective scheme of public education, not by indirection, but frank, fearless, and in the open.

Our Courts, public officials, and professors and students of economics should be included in the classes of those to be educated. The conflicting and often faulty opinions delivered by our Commissions and our State and Federal Courts demonstrate that we of the laity must not submit too subserviently to the opinions and decisions which are quoted for our control. We may have to submit temporarily in cases at issue, but we should never surrender our beliefs where we know that we, as engineers, are more completely and accurately informed than the theorists who attempt to control us. We must patiently, persistently, courageously, and openly combat the theories which we believe to be founded in error or on an incomplete survey of all the questions involved.

In order to meet those who are thus engaged in controlling the situation, however, we must be more at one among ourselves. The fact is that on some of the questions at issue engineers are not in complete agreement.

Thus the Commissions and Courts are given the opportunity to choose between opinions at variance, and so are encouraged to pick and choose, item by item, until there is consolidated all possible features which are unfavorable to a fair solution of this valuation problem.

Another obstacle to a fair and consistent system of valuation is pointed out by Mr. Wilgus: the widely varying laws of the several

Mr. Humphreys.

States, not to speak of the widely varying interpretation of these, often ambiguous, laws.

Unquestionably, Mr. Wilgus is correct when he states that, as a general proposition, valuations cannot be determined accurately and fairly through reference to the records and books of account extending back through the years. It is not difficult to explain, to those qualified to understand, why this is the fact—but the fact has to be met, apart from the question of an adequate reason therefor.

Nothing could be further from the truth that, because a railroad or any property of magnitude can be inventoried, even if accurately and completely inventoried, the valuation can be determined by the inclusion of the items thus inventoried. Omissions and contingencies are not thus covered. Surely no engineer who has had any broad experience in construction could honestly so claim.

It is often the contingencies, lost to sight in the completed work, which add materially to the cost, and this outside of such overhead charges as preliminary or initiation expense, interest during construction, administrative expense, cost of procuring capital, etc., etc. For instance, if an appraisal should be made ten years from now of the New York Central Terminal, would an inventory of the existing structures disclose the legitimate and necessary cost of this great undertaking? This point is well developed in this paper.

One of the most difficult features involved in public service appraisals is that of depreciation. This subject is misunderstood by many who claim the right to speak authoritatively thereon.

If the accounts are accurately kept, so that proper discrimination is maintained between charges to construction and to maintenance, no deduction should be made for depreciation from cost to reproduce new.

Troubles in this matter have, no doubt, in part been occasioned by the more conservative practice of late in keeping accounts so as to spread the cost of final renewals ("depreciation") more uniformly over the period benefited. This leads those on the outside—and sometimes those on the inside—to think that we should deduct the assumed or estimated accrued depreciation from cost of plant. A depreciation reserve is required only to point out to us year by year that there may be certain charges against income not shown by the current expenditures and so warning us not to over-estimate our profits; but, if final renewals have not been cared for completely in the current expenditures, then there is the liability therefor, resting against the proprietors, which must be met when parts of the plant come to be renewed. All repairs and all renewals, whether current or deferred, are chargeable against income, and so must be met by the rates. In the case of a public utility, adequately maintained and rendering

efficient service to the public, to deduct for accrued depreciation according to the practice of some of our theorists, necessarily results in confiscation of investment. Mr. Humphreys.

Certain appraisers of reputation have taken the ground that if, for instance, railroad ties have an effective life of 10 years, and one-tenth of the total number is renewed each year, there is no need for a depreciation reserve, but the value of these ties as a whole should be depreciated 50 per cent. Nothing, according to the writer's way of thinking, could be more fallacious. The property, as far as this item is concerned, is thus maintained at maximum efficiency. What more could be done? Why then should one-half of this portion of the investment be confiscated? If so ordered, then we should be permitted to include a like amount as one of the overhead charges. If such an elimination is a necessary feature of railroad operation, then this is a legitimate item of cost.

Those who have not studied this subject carefully seem to think that because we should expect to allow, if we were selling, and should expect to claim, if we were buying, an allowance to be deducted from the purchase price to cover estimated accrued depreciation, therefore depreciation should be deducted in an appraisal for rate-making. On the contrary, the reason for such an allowance for depreciation in case of purchase and sale, furnishes the reason for not deducting it from an appraisal of cost new; for this indicates that there is the assumption on the part of the purchaser of a liability resting on the present owner. This is well set out in Mr. Wilgus' concluding paragraph.

This feature of depreciation the writer has treated at length in a recent paper.*

In conclusion, the writer desires to state that we, the Engineers of the United States, must appreciate that on us rests the grave responsibility of protecting the innocent investor, the widow and orphan, from spoliation at the hands of honest and dishonest interpreters of the laws of our country—Federal and State.

J. H. Gandolfo, Assoc. M. Am. Soc. C. E. (by letter).—The problem of obtaining the physical valuation of the railroads of the United States is probably one of the most important relating to public service corporations and properties that has ever been undertaken; and the question of rate-making, whether founded on the results obtained from physical valuation or not, is dependent on so many questions, both economic and financial, as to make it very difficult to discuss the matter within the limits of a single article. Mr. Gandolfo.

To investigate this matter of valuation and rate-making properly, it is necessary to study the historical development of the railroads from

* "Depreciation: Estimated and Actual," read before the Institution of Gas Engineers of Great Britain.

Mr. Gandolfo. their beginning, and then to analyze the entire question according to the principles of economics. This necessitates looking at the problem in a way very different from the generally accepted financial viewpoint.

Mr. Wilgus attempts to show that, from every point of view, the only just and equitable valuation to be placed on railroads for rate-making purposes, is that of replacement value new; but are his premises correct in this argument, and is replacement value new the logical figure to be used for this purpose? Would such a basis of calculation be just and equitable to the railroads, as well as to those who must pay the rates?

When a physical valuation is undertaken, no matter for what purpose, there is no doubt that the replacement value new should be estimated, taking the average cost of materials and labor for the five previous years, as is usually done; but the actual cost to date should also be determined, by reference to records which may be available, or, if not, by reconstructing all conditions as they existed at the time of the building of the road. The depreciation of the various elements entering into such a property should also be estimated at the same time. These three items can thus be determined simultaneously at very little more cost than that necessary to obtain any one of them alone, and at infinitely less cost than would be necessary to obtain each one separately. In fact, it may be said that for all practical purposes it is necessary to obtain all three of these items as a basis for study and comparison.

Referring to the sale value of the various elements entering into the construction of a railroad, such valuation is productive of no results except, perhaps, as a very poor, makeshift basis for taxation. If quantities of materials such as go to make up a railroad, were suddenly thrown on the market, it would in most cases at once depress the second-hand price of each, and thus create a price far below even the ordinary sale value. On the other hand, even taking the market sale value for estimating purposes would, as Mr. Wilgus points out, by no means give any idea whatever as to the actual values or actual costs of railroad property. As for the sale value of a railroad as a whole, such a quantity may be said not to exist, as there are no buyers for such properties in the open market. When such a property has been offered for sale, it has been bought in by a syndicate composed more or less of interested parties, and at its own price.

On page 205, the author states that it would be a very difficult matter to get any idea of the original cost of the railroads on account of the non-existence of records on the subject, and gives this lack of records as an argument as to why original cost to date should not be obtained and used. The mere difficulty of arriving at a result

is surely no excuse for not doing the work. It is only a matter of more time and patience, to go back and, with figures based on known prices for labor and materials of the period, reconstruct a road built fifty or more years ago, than it is to reconstruct the road under present-day conditions. Mr. Gandolfo.

Also, at various places, the author mentions different items of expense in the planning and construction of a railroad, which he seems to fear might be omitted in a physical valuation. On page 206 he says: "Reconnaissances, preliminary surveys, and estimates of cost, revenue, and profits are needed * * *", and, on page 211, various construction items are referred to as matters which should not be overlooked. It is difficult to conceive that any one engaged in a valuation or appraisal of a railroad, except the merest tyro at the business, would fail to take cognizance of such items. The cost of all preliminary work, of whatever nature (including promoting), and the cost of all construction work, should be estimated, no matter for what purpose the valuation is being made. One might as well try to estimate the cost of concrete in place, and say the cost of the form work should not be taken into account.

When it comes to including in the physical valuation the cost of what Mr. Wilgus calls "the educational and development stage", however, the writer fails to see why such expenses should be estimated for this purpose. This matter is purely one of operation, and the obligations which it was necessary to issue in order to obtain working capital in the early stages of a railroad, should have been paid off from profits, and a fund kept for this purpose from accrued profits. On page 207 the author says, "under the most favorable auspices, it usually lasts for several years". If a railroad is properly run and properly managed, the educational and development stage never ceases. For instance, the training and education of employees for their various duties never end. The study of places where economies can be practiced is something which should be in the mind of every one connected with a railroad at all times. The attempt of the advertising department to induce more and more travel, both freight and passenger, to patronize a given road, is also a never-ending work. As far as "errors of design and construction, as developed by operation," are concerned, these should be paid for out of profits, if there is money to pay for the work, because there is practically no limit to which such changes can be carried. To-day, expensive tunnels, long cut-offs, great trestles, deep fills, and heavy bridges, are being constructed on various railroads, simply to correct just such errors of design and construction; and if such corrections and betterments are to be capitalized as new development, what security is left for the obligations already issued against the original and now abandoned work? Of

Mr. Gandolfo. course, it must also be taken into consideration that improvements are often made for the purpose of handling increasing traffic and steadily increasing weights and loads of rolling stock. In all such cases, the only thing to do is to take the difference in cost between the old work and the new (if the cost of this new work is greater), and charge it to physical valuation. To illustrate this by a concrete case, assume a bridge, the original cost of which, complete, was \$100 000. Now, assume that this bridge is replaced by one costing \$200 000, either to correct some error in original design, or to provide for more or heavier traffic. Such a structure must be included in the valuation at its cost of \$200 000, or \$100 000 added to the original valuation of the road; but by no means should the total amount of both structures, or \$300 000, be included in any valuation, or obligations issued against this latter amount, as has so often been done in the past. Thus, it can easily be seen that the engineer who attempted to include any such feature as this, of the educational and development stage, in a physical valuation, either for original cost to date, or replacement value new, would be confronted by a peculiar problem. He would not know where to begin, or where to leave off. It would be simply a matter of personal opinion as to what and how much to include to cover some arbitrary period.

Furthermore, in this connection, it is to be remembered that, like everything else, railroading has been a slow growth, and that it has not been necessary to organize forces from absolutely ignorant material to handle the complicated equipment and traffic of to-day. The transition from the small systems beginning in England in 1825, with a few miles of track, equipped with tiny cars and 8-ton locomotives, to the great transcontinental lines of to-day, with thousands of miles of track, cars with a capacity of 110 000 lb., and great Mallet compound locomotives, was not made in a single stride. It was a slow development, and the operating force was gradually trained and organized to meet the conditions as they arose. And, further, if it could be imagined that a great railroad system was to be created new, to-day, with miles of track, modern equipment, and great terminals, even then, it would not be necessary to organize a new force from raw material. All the men holding positions of any moment would be obtained from other railroads. A few years ago, the writer was connected with a new railroad while the operating force was being organized, and all the men in positions of any importance, including such as general superintendent, shop superintendent, heads of all operating divisions, etc., and, also, in many cases, the subordinates, were obtained from other roads.

In regard to cost of reproduction less depreciation, it does not seem as if any allowance should be made for depreciation, either in

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an original cost-to-date valuation, or cost of reproduction new, assuming, of course, that the road has been kept in first-class condition; that an amortization fund has been provided for, to meet all maturing obligations; and that some provision has also been made for obsolescence. If such items have not been fully provided for, then the question of depreciation may become a very important one, and must be given very careful consideration, with special attention to the object for which the valuation is being made. This can be made clear by referring to the coal pocket mentioned by the author on page 208. If a physical valuation had been made of this property the year before it was to be renewed, and if this item had been included at its full value of $600 000, there must have been one of two provisions made, as follows: Either an amortization fund must have been created, by which all stock, bonds, or other obligations, representing the full value of the coal pocket, will be retired at the end of the next year, so that new obligations can be issued to pay for the new work; or, if the obligations against the full value of the coal pocket are not to be paid off, there must be some fund, which, at the end of the next year, will amount to $600 000 in cash, so that the new coal pocket can be paid for without issuing any further obligations. Otherwise, if no such provisions have been made, the coal pocket must be included in the valuation simply at its present value.

This case illustrates another method which may be used to estimate depreciation, and that is, when the physical valuation is being made, to estimate the amount, in a lump sum, which it would take to put each item in first-class condition for the uses for which it was intended, and then to deduct this amount from the actual cost or replacement cost, whichever plan is being followed. This method of estimating depreciation has its advantages in that it eliminates assumptions and estimates, as to the probable life of a structure; but, on the other hand, it has its disadvantages, inasmuch as it only applies strictly to present-day values, and takes no account of the fact that depreciation is apt to be more rapid in the later years of the life of any structure. This is one of the reasons why the writer stated in his opening paragraphs, that values for depreciation should be determined for purposes of comparison and study.

On page 209, in discussing land values in regard to "original cost" valuation, Mr. Wilgus says, "it ignores the increment that is enjoyed by all other property owners through increase in the population and prosperity of the country"; but, in this connection, Mr. Wilgus seems to ignore the following fact. A railroad, or, in fact, any other public service corporation, does not occupy the same position, relatively to the public, or to the State, as a private citizen. A corporation has no being and no existence until created by the grant-

Mr. Gandolfo. ing of its charter by the State, which charter defines in just what direction the corporation is to exercise its functions. A railroad corporation is thus granted rights, privileges, and immunities by the State, which are not accorded to the individual. It is a fundamental axiom of both law and economics that something cannot be obtained for nothing, and thus, in return for these grants, the railroad must give up some things to the State, which the latter cannot demand from the individual. In regard to this question of land, it must be remembered that, especially in the early days of railroading in the United States, large grants of land were made to these corporations by the State, either without any compensation whatever, or at a merely nominal figure. It is not equitable that the present generation, having lost this land through no fault of their own, should now in addition be indirectly taxed for what their ancestors gave away without compensation.

In the following paragraphs the writer wishes to point out some additional anomalies and contradictions which would occur if the replacement value new was used as a basis for rate-making purposes.

According to an old print, the great Tring cutting, made in 1837, on the London and North Western Railway, was excavated by hand, and the spoil was removed in hand-barrows, guided by men, and pulled up inclines on the sides of the cutting by ropes passing over pulleys and worked by horse-power. In estimating such a piece of work for replacement value new, should these conditions be duplicated, only with present-day prices for labor and material, or should modern steam-shovel conditions prevail? It seems to the writer that, in a case like this, the railroad might find that, in spite of high labor and material prices, the cost of such a work would be much less than the original figure, and thus be an injustice to the company.

Referring to the temporary trestle work mentioned by Mr. Wilgus on page 211, on many of the early roads in the United States such structures were built of timber obtained along the line. It surely would not be just to estimate a value for such a structure based on present-day figures for the labor and materials entering into it.

About seven years ago, the cost of a certain class of steelwork, erected in place, was 4.07 cents per lb. Since that time this price has not been reached, but the same class of work has been done as low as 2.87 cents per lb., or a difference of 29½ per cent. It would not be just to say to the railroad which had paid the higher price, "Here, you must reduce your capital, because the work can now be done cheaper". This money was honestly invested, and it would be unjust to wipe out a part of such invested capital.

Another objection against replacement value new for rate-making purposes, is that it would be a constantly varying quantity, whereas

original cost to date would only have to be corrected for such additions and betterments as were added from time to time. Mr. Gandolfo.

In conclusion, it seems to the writer that for rate-making purposes, the total cost of the work to date, is the figure to be obtained. This figure represents the actual amount of capital invested in the enterprise. It may be argued that some railroads have depreciated, instead of appreciated, in value, and that no rate could be fixed, which would give a fair return on the investment. This is true, but in such a case the enterprise is not a success, the company is insolvent, and should be treated as such.

It may also be argued, as Mr. Wilgus indicates, that this method of valuation puts the older roads in a much more favorable position than those built at a later date. This is also true, but it was a question for the promoters and builders of these later roads to consider in their preliminary estimates, not for the State and the people to consider now.

T. Kennard Thomson, M. Am. Soc. C. E.—The Society, and indeed the whole country, owes Mr. Wilgus a vote of thanks for his very complete and valuable paper. It would be difficult to find a phase of the subject on which he has not touched. Mr. Thomson.

The valuation of railroads is, of course, one of the most far-reaching and important investigations ever undertaken by this or any other Government, and the results obtained by the present Commission will naturally be of inestimable value. It is safe to say, however, that, unless this work is done in harmony with, and with the hearty co-operation of, each railroad, the results will not be worth one-quarter of what they should be.

Although the speaker is a strong believer in the benefits which will accrue from the present Government investigation, he thinks that the ultimate object is the "intrinsic value" of the railroads; that is, the real commercial value—not an artificial assumption.

If desirous of buying anything, one must pay what the seller can be persuaded to take, and he, naturally, will not sell if he thinks he can get a better price elsewhere. A seller likes to think that he is getting back all that was spent on his property plus interest, either simple or compound, but he would probably sell at once for a small fraction of the cost, if satisfied that the proceeds could be invested so as to net him a larger or safer income.

The value of a railroad, like that of any other property, must fluctuate enormously from time to time, due to changed conditions or changed management; and these fluctuations may have very little relation to the original cost of the property.

In buying a fine diamond, does the purchaser inquire what was spent in mining it? If two young men take the same course at

Mr. Thomson.

Harvard, one at a cost of a few hundred dollars a year—which he earns as he goes—and the other at a cost of many thousands of dollars, will not the first almost invariably be able to sell his services for a higher rate? If these two boys were to invest the same amount of money in houses, one might build or buy a number of houses which he could sell for a big profit; the other might put all his money in a single house and not be able to sell it for one-quarter of its original cost. Two men might start a grocery business, under exactly the same conditions, but with entirely different results, as all know, and so on, *ad infinitum.* Why, then, should a railroad alone be forced to put false or fictitious values on its property—for sale, rate-making, or taxation, etc.—instead of its real value at that time?

Suppose a railroad is only a couple of hundred miles long. It may be in first-class condition in every respect, but if it has not a working arrangement with other roads at each end, whereby it can obtain its share of freight, its earning capacity will not be enough to pay operating expenses; and to-morrow a new manager may "get the business" and pay the stockholders a handsome profit. This has been done over and over again. In one case a young engineer, who saw a western road which was "run down", told the owners what he could do if they paid him a percentage of the profits; his proposition was accepted, and this made him a multi-millionaire. How would any basis of actual cost, or cost of reproduction, be applied to these cases?

Mr. Wilgus very truthfully states that it is impossible to ascertain the real cost of a railroad already built. He also states that it has never been possible to predict the actual cost of a road; "cost of reproduction" would come in the same class.

Among other things which have cost the railroads much and would be hard to find are the items of ballasting mentioned by Mr. Wilgus, who states that 24 in. of ballast are needed. In many cases the depth of the ballast is actually more than 12 ft.

Cost of litigation, and other obstructions by property owners, are enormously expensive, not only directly, but also in the brain energy required to overcome them, which might have been spent on the construction or operation of the road. For instance, the Chesapeake and Ohio Railroad, being unable to obtain from an old Virginian permission to use part of his property, took possession. The owner removed the tracks and buried the corpse of a woman there; the railroad company dug up the body and buried it 150 miles away; the owner found the body, brought it back, and again buried it on the "right of way", this time embedding it in concrete. As the concrete was very hard, and the railroad company did not like to use dynamite, it simply raised the grade of the track, and allowed the body to remain in peace under the rails.

Without the co-operation of the railroads, an investigator might easily overlook the old New York Central Railroad line through Kingsbridge, New York City, and estimate the Marble Hill cut-off at a fraction of its cost. In the West a railroad tunnel which had been closed three times by land slides was abandoned for a temporary line with a very sharp curve, and years afterward was rebuilt in a different location. Mr. Thomson.

But why continue when every one knows that the exact cost of an old or to-be-built railroad can only be ascertained approximately?

Mr. Crehore's suggestion, that the railroads put a valuation on their own property, has much merit, and reminds one of the method used in Germany for many years, the speaker believes, successfully, whereby the owner of a house gives the valuation of his property, and this is used by the Government for purposes of taxation and fire insurance.

CHARLES S. CHURCHILL, M. AM. SOC. C. E.—In the speaker's opinion, Mr. Wilgus should be thanked for having brought out his points in a very clear form, these points being recognized as generally correct. Within a few weeks after the issuance of the paper the Government officials made some very important statements: First, that the valuation of properties of common carriers in compliance with the Act, Section 19-a, is an original proposition; second, that it is a continuing one; and, third, that the physical elements must be compiled or estimated in accordance with the Interstate Commerce Rules of Accounts, as stated in the Act. Mr. Churchill.

Under the first statement, that the valuation of properties is an original proposition, it appears probable that no value which has heretofore been made will fully meet the requirements of the Act. Under the second statement, that the requirement is a continuing one, it becomes extremely important that accurate records shall be kept of the cost of all work, in accordance with the Interstate Commerce accounts. This statement also has led to plans being formulated by Government officials for preparing their files to cover 100 years. The third statement, that all records and estimates shall accord with the Interstate Commerce accounts, is one of the most important features of the whole Act, because this simple requirement answers many of the points raised in the written discussions on this subject, especially for the reason that the Interstate Commerce accounts are quite clear. Therefore, it is thought very important that the law should be studied very carefully, as it is a good one in many respects.

Many progressive railroad properties double themselves within 20 years; therefore, suppose one makes a valuation now, 20 years hence very little trace of the items reviewed at this date will be found, because the properties are changing and building up constantly.

Mr. Churchill.

The question has been asked, "What has the public gained in existing railroad properties"? Recently, some records of railroad construction between 1850 and 1856 have been found, which show that a railroad was being built across one of the Colonial States, and that within 6 years—before it was entirely completed—the actual returns in taxable values were doubled in each of the counties through which the railroad passed. This old record shows where both the individual land owner, and also the communities, counties, and State have gained from the fact that certain individuals combined in constructing railroad properties. This is one kind of return out of many others which have come to every community and State in which railroads exist.

Attention is called to the point in the law, that the valuation asked for is not confined altogether to physical features; also, that the law specifically calls for the cost of the property to date; in other words, that the investment in a property must be determined as closely as practicable, as being an important element. If, therefore, a railroad has built a property within recent years, or has bought one at an earlier date, and has made considerable additions thereto since the date of acquirement, the original cost plus the additions must be determined. In most instances records of expenditures have been carefully kept, and usually there are three parties to these records, namely, the railway company, the contractor, and the engineer. The amounts paid, therefore, show on their face the true cost of the work when built.

Finally, attention is called to the recent decision of the Supreme Court in the Minnesota Rate Case, wherein a favorable decision was given in one case, and, in that case, the original investment was protected.

In conclusion, it seems important to draw attention to the greater likelihood of overlooking items of cost after the structure or railroad has been completed than in making proper allowance for contingencies before the work is undertaken; and that these items of cost, which are so likely to be overlooked in works already completed, should be searched for in great detail, so that the application of percentages to cover past contingencies may be avoided as far as possible.

Mr. Coombs.

R. D. Coombs, M. Am. Soc. C. E. (by letter).—It would appear to be necessary to have a more general acceptance of what is meant by "cost" and "value": Whether the "cost" of construction is its actual cost, what it should have cost under actual conditions and average good management, or what it should have cost under average conditions and average good management; and whether the value new corresponds with any or none of these.

Assuming that the railroad's actual expenditures could be determined, they would in fact represent the cost, whether that cost

was unusually low, or unusually high, unless it is accepted that deductions should be made for bad luck, ignorance, or dishonesty. Mr. Coombs.

If it is a fact that, owing to working conditions, weather, accidents, high wages, and relatively expensive material, a certain work cost the railroad an amount greatly in excess of its probable cost under average conditions and efficient management, should the actual cost be accepted as its value new?

If the necessity of maintaining service compels expensive construction, should the excess cost over that of average conditions remain a part of the cost of the work, and would that cost be accepted as the value new?

If the improved methods of construction now available would decrease the cost of new work below the amount actually expended, should such methods be used in estimating the value new?

If the original contour and the topographical conditions along the line are assumed to be restored in the cost of reproduction new, a fair estimate, in some cases at least, would require us to assume the surrounding country restored and to make some allowance for working under such conditions.

Until those discussing this matter agree on the foregoing points, there will naturally be no agreement on further extensions of the subject.

That the actual cost would be difficult to determine does not appeal to the writer as a logical reason for discarding it altogether. In many cases the cost of reproduction new might be used as a "side light" on the actual cost.

It must be admitted, however, that the actual book records are not absolute cost records. In fact, the writer does not believe that the accounts of any railroad, and of but few contractors, would show the complete expenditures for a given construction.

Referring to the acceptance of one basic principle or another, and to the deduction or inclusion of depreciation, depending on the purpose of the valuation, it would appear:

First.—That if the directors have set aside a depreciation fund, that fund plus the physical property represents accurately the full physical value of the property, and may be properly used as the physical value for any purpose.

Second.—If such a fund has not been established, the physical value is represented by the actual physical property, and is less than 100% of its book value.

Third.—As the present stockholders are not necessarily the original stockholders, some of them, under the last assumption, are innocent purchasers of stock which does not represent full value, in so far as the physical property is concerned.

Mr. Coombs.

Fourth.—The physical valuation of a railroad is its valuation as a railroad in its present location, and, from an engineering standpoint, the value should be the same either for sale, taxation, or rate-making.

Fifth.—If taxes are too high or too low, change the percentage; if freight rates are too high or too low, change the rates.

Mr. Ingersoll.

COLIN M. INGERSOLL, M. AM. SOC. C. E. (by letter).—To railroad engineers who have advanced in their profession, and have been through the construction period from beginning to end, this paper will most certainly appeal, for it describes in detail the very steps taken in building a railroad.

As yet, the physical valuation of railroads is in its infancy, and no definite method has been determined for making such valuation; but Mr. Wilgus has done a great service to the Profession in publishing his paper, and this will be proven when the final method of valuation is determined by the Government.

The method to be used in arriving at the valuation depends somewhat on what use is to be made of it. If it is for a quick transfer by sale, it would seem that the property should be turned over on a reproduction value, less depreciation; but if the property is kept up in a substantial manner, it would seem to the writer that the cost of reproduction new is the method which comes nearest of any that has yet been advanced to putting all roads on the same footing. The original-cost-to-date method of valuing would not seem to the writer to be fair for all, for until very recently the railroad companies were rather slack in keeping accurate costs of work. Take a railroad built in the Sixties or Seventies, the books of which have been kept up (and the writer doubts if there are very many such roads); the books would show the money that had been put into the railroad, but without any allowance for the enhanced value of the property. Perhaps the road adjoining has kept no books, and, consequently, in order to arrive at the present value, some method of reproductive value must be used.

In the first instance one cannot, by any arbitrary rule, arrive at the present-day values unless one gets at the cost of reproduction; and if this is done, the original cost is of use as a side light. Then, again, rarely in "original costs" are the whole costs shown. In construction work contractors have often received extra compensation, either by agreement or through legal process, which does not appear in the book cost of the work. Book costs show the cost of the property, but rarely include all the costs, such as engineering, legal fees, commissioners' fees, interest on money during construction, and operating losses, which really are a part of the cost of the property. In many instances land has been given and contributions have been

made, for establishing stations, side-tracks, etc., in order to induce the company to build. It would seem to the writer as if they were just as much a part of the road that should be allowed to earn interest as that which was paid for. It is taxed, and can be sold and credited to the company. In many instances the purchase price of land is more than the value of the adjoining land, and rightly so. Those who have had to do with the purchasing of right of way know that one cannot go diagonally through a piece of farm land, say 5 acres, take out 2 acres, and leave the remainder of the same value per acre. For that very reason an extra price has to be paid, whether it is called an increment, damages, or an inducement to sell. It is there, and is a part of the cost of the property. Mr. Ingersoll.

If for any reason the records of the purchase of this piece were destroyed, it would not seem to be fair that it should be valued on the basis of the value of the adjoining property. It is true that railroads can condemn, but, with small pieces of land, it is generally cheaper to purchase at a higher price than to go through condemnation proceedings, pay for commissions, lawyers, engineers, and witnesses, which, if the property is not expensive, amounts to a big percentage of the cost. Take the same case of 5 acres of land, at, say, $10 per acre. The owner for some reason, often for gain, refuses to sell. Has it been the experience of any one who has purchased land for right of way that the railroad company can get the 2 acres of land needed for $20 by condemnation? No; the lawyer, the commission, the witnesses, the engineering, etc., would cost more than twice that amount, and some method must be found to include these legitimate expenses in the value of the land. Otherwise, the railroads will have to put in capital on which no return in the way of dividends can be paid. This certainly would not seem to be fair to the investor.

The writer believes that it is proper to charge property with its share of the engineering and legal expenses, and contingencies, for the very good reasons set forth by Mr. Wilgus.

For rate-making purposes it is not believed that depreciation should be deducted from cost of reproduction. Depreciation is an element of operating costs and an obligation of the stockholders.

If a railroad has been allowed to run down while dividends have been paid, then the stockholders should forego dividends, and the earnings should be put into the property until it is brought up to standard.

Arthur M. Waitt, Esq. (by letter).—The writer has read this able paper with great interest, as it is based on large practical experience, and deals in a broad and comprehensive manner with a subject which is of vital importance both to the railroads and the general public. Mr. Waitt.

Mr. Waitt. Probably few, if any, men have given deeper or more conscientious thought to the subject, or have had such an extensive practical experience to enable them to deal with the matter as fairly and comprehensively, as Mr. Wilgus.

It is the writer's firm conviction that in determining the physical value of railroad properties for purposes of rate-making, in justice and equity, only one basis can be used, namely, that which considers the actual capital required to reproduce the conditions and facilities found necessary by the railroads to produce their revenue, and he fully agrees with Mr. Wilgus that for rate-making purposes depreciation should not be deducted.

An intimate relation with the shop equipment and rolling stock of both American and European railroads for a long period has caused the writer to study especially the treatment of these important elements in the total valuation of railroad properties.

Oftentimes, in his practical experience, the question has arisen as to whether comparisons of the values of equipment and rolling stock taken at different periods, several years apart, would show an appreciation or a depreciation. In every instance, a careful analysis of the facts and figures, ignoring new equipment charged to the "improvement" or "capital" accounts, has shown no net appreciation in values, due to the fact that new tools and rolling stock purchased to replace old ones were invariably heavier and more valuable than the worn out or obsolete equipment. It would be true only in a few exceptional cases that, in valuations made 5 years apart, there would be an actual depreciation in total values of unincreased equipment.

If physical depreciation were deducted from cost to reproduce new, figures would be obtained which would not fairly represent the capital involved to provide the facilities and operating conditions on the railroads, for which adequate rates for transportation should be allowed.

A fair valuation of equipment or rolling stock, or, in fact, of any of the elements composing the railroad's total value, cannot be made, except by a reasonable personal examination by qualified experts in the different fields, and it cannot be done equitably by arbitrary or set rules applied regardless of local conditions.

In a valuation of the equipment or rolling stock of a railroad to determine its purchasable value at any given time, different principles would apply, and in such cases the cost of reproduction new, less existing depreciation, would be the correct measure of value. In a valuation for this purpose, no set prices and rules, applicable to all cases, can be established; the prices and percentages must be established after an examination of a sufficient portion of each class of equipment to enable an expert to determine the comparative initial values and the quality or standard of maintenance, this latter to be used in the case of each class of equipment or rolling stock.

It would be manifestly unfair to value by the same standards and figures the property of two railroads, one of which constantly purchases new locomotives, cars, machinery, and tools of the highest grade, having all the best features for obtaining high efficiency in service and longevity of equipment, and on which, by the wise expenditure of money, the equipment is maintained in the best condition for service, while the other road purchases cheap equipment and expends as little as possible in maintenance. Situations approaching each of these cases will be found, and the writer maintains that they can only be handled equitably by qualified expert judgment after personal inspection. Mr. Waitt.

No more important facts can be urged for consideration and correspondingly wise action than those which are made prominent in Mr. Wilgus' paper, but in closing the writer wishes again to emphasize two of them which, to his mind, are the most important, namely:

1.—The cost of reproduction new without depreciation should be taken as the only fair and true measure of the physical valuation of railroads for purposes of rate regulation.

2.—Expert skill and experience must be enlisted in order to make a proper estimate of the true value of railroad properties.

Stevenson Taylor, Esq. (by letter).—The general principles involved in covering methods of determining valuations of railroads, with the attendant matters of bookkeeping, establishment of costs, and the important subject of depreciation, have been considered by the author with admirable thoroughness, and the writer has no good reason for differing from his conclusions. Mr. Taylor.

If we hope to encourage "men fitted by experience, acquaintanceship, resourcefulness, courage, and tact," as well as investors large and small, so that the building, extension, and upkeep of railroads may proceed further to advance the interests of the whole country, we must deal justly with this broad question of public utility and with those who are and who may become directly interested therein.

The author has pointed the way. There may be differences as to minor details, but his paper covers the subject in a masterly manner and with fairness and justice to all concerned, including the general public.

F. C. Hand, M. Am. Soc. C. E. (by letter).—The valuation of the railroad properties of this country under the recent Act of Congress is indeed a Herculean task, probably the greatest that has ever been undertaken by our Government. The results of this work will be far-reaching, in whatever light they may be considered, whether for purposes of taxation, rate-making, ultimate Government ownership, or for the purpose of regulating stock and bond issues, as is now the case in Texas. Mr. Hand.

Mr. Hand. The writer, however, cannot agree with the author that the ultimate valuation shall depend on the purpose for which it is to be used. There can be but one measure of actual value, as the Courts have said that the value of a property depends on the use to which it is put, and railroad property can certainly only be used for railroad purposes. If a valuation is placed on the property for rate-making purposes, where is the injustice in requiring taxes to be paid on the same valuation? Taxes are only one item of expense in the operation and maintenance of a railroad, and are taken into the accounts as such and are considered in fixing rates.

The sale-value method would hardly seem to be worth considering in this connection, as so many questions of franchises, connections, traffic facilities, outstanding obligations, and many purely intangible elements of value, that might and usually do have weight with the purchaser, would enter into the transaction as probably to modify the actual physical value very materially.

Should the Government ever acquire the roads, the owners would insist on being paid the value to them at the time of purchase, and would probably also insist on capitalization value in most cases. In that event, the valuation to be made by the Interstate Commerce Commission would be most valuable, as the relations between actual values and capitalization will be clearly defined in every instance.

The Act of Congress provides that Original Cost to Date shall be determined as one of the alternative plans, and, as Mr. Wilgus truly says, this will be a difficult undertaking. Simply because it is difficult does not by any means make it impossible. It is probably true that many of the old records are very incomplete, and also that many of them are missing, but all engineers know that in almost any appraisement there are some missing and incomplete records, and that it is necessary to go back to the period of construction for cost data, and that the greatest care is also necessary to estimate properly all conditions existent at the time when the work was actually done. This will be at times extremely difficult, but with care it should not be impossible.

The writer remembers a case in point: Some years ago a line of railroad was built across a marsh for about 3 000 ft. There was no pile-driver available, and the line was cribbed with timbers cut from and adjoining the right of way, a very expensive operation. As soon as the track was laid a top-driver was obtained and a pile trestle was driven. A few years later the marsh was drained and the entire trestle was filled with steam shovel, and now there is no indication, except the absence of borrow-pits, which would put an experienced engineer on inquiry, to show but what the entire fill might not have been put in at the time the road was originally built. Possibly the original records in this case are available; if they are not, careful in-

quiry and investigation would detect the original conditions, and the original cost could be arrived at pretty accurately. Mr. Hand.

Original Cost to Date is evidently intended by the Act to show the amount of capital actually invested in the properties, and if all the items of original cost are included the result should be very satisfactory.

The author's ideas on Cost of Reproduction New are extremely lucid, and would probably be accepted by most engineers of experience as giving a good definition of the various steps, except possibly the "educational and development stage," which is not a closed chapter when the road becomes a self-sustaining or dividend-paying proposition. This system of education and development is never completed, but the cost, of course, is a part of operating expenses.

The method of valuation by Cost of Reproduction, Less Depreciation, which is provided for by the Act, is probably the one of the three methods which will be productive of the most discussion, and the one on which the greatest variance of views will appear, except possibly on the question of estimating proper land values. This extreme variance of opinion appears to the writer to be chargeable to the effort to establish a fixed basis or standard by which depreciation shall be computed. It is of course beyond dispute that nearly all the essential parts of a railroad are constantly wearing out, and consequently depreciating, but constant repairs and renewals are being made, paid for out of operating expenses, so that a first-class railroad may at all times be said to be nearly 100% perfect, from the standpoint of efficiency. In such a case, depreciation is surely negligible, as an amortization or surplus fund is being constantly set aside to provide for renewals and also for obsolescense. This fund, by whatever name it may be called, is provided out of earnings, that is, it is being provided by the public, and its reinvestment should not be taken into capital account.

Unfortunately, not all roads have handled this matter in this manner. Some of them have paid to their stockholders, in dividends or otherwise, the money which should have been set aside, and have floated another bond issue when extensive renewals or improvements became necessary, and other roads have never been able, from one cause or another, to earn enough money to set aside anything for these purposes.

These are the actual conditions as they exist on our different roads, and the writer can see no way, except by a careful and painstaking examination of each particular case by itself, to arrive at a fair estimate of depreciation. No general formula can be adopted which will be fair in all cases.

The question of land values is also one on which there are many varying views. The author says:

"The 'original cost' measure of land values is believed to be unfair, for two reasons: It ignores the increment that is enjoyed by all other property owners through increase in the population and prosperity of the

Mr. Hand. country; and it would lead to the inconsistency of low original values on lines unfortunate enough to have preserved their land records, and high costs on adjacent lines where the absence of records would make imperative the use of present-day values."

The writer is unable to see anything really unfair in this, as under the provisions of the Act both the Original Cost and Cost of Reproduction New are to be ascertained. No increment could be claimed under the first, as the purpose is to ascertain the original cost only; and, in case the company's records are defective, a reference to county records will generally give the desired information. In the second case, the present-day values can be readily found, although there are many different plans suggested for arriving at this. The writer believes that this too should be the subject of a careful painstaking investigation in each individual case, and that no rule can be fixed which will be at all fair to both the railroads and the public. The right of way through each tract of land should be considered as a separate unit, and the matters of severance, plottage, and other damages, should be considered as they affect each tract. No uniform percentage can be used which will reflect actual values accurately, for perfectly obvious reasons. The writer has known of many instances where rights of way were obtained through comparatively large tracts when only actual acreage values were asked. The lands were not badly cut up; cattle passes or overflow waterways gave ready passage from one side to the other, and no damages beyond the acreage value could be shown. On the adjoining tract, however, the situation might be quite different, but simply because damages were caused in one case in a sum perhaps more than equal to the value of the land, is no reason why a percentage should be added in the other.

In this paper the author has not considered the capitalization of the railroads as a factor in ascertaining values, but the writer feels constrained to add a few words on this phase of the subject, as he believes it is a very live question and one that will be brought to the attention of the Valuation Division many times before this work is complete. In fact, considerable stress was laid on this feature in many of the discussions in Congress during the pendency of this Act and in the discussion incident to the passage of the appropriation for beginning the work.

The pertinence of a discussion of this feature will be apparent in the event that, as a result of this Act, an attempt should be made to regulate the issues of stocks and bonds, or should a proposition for Government ownership be seriously considered.

There is a pretty widespread impression that there is an enormous quantity of water in railroad securities, although this is stoutly denied by the railroads and by such statistical authorities as Mr. Slason Thompson.

Mr. Hand.

Mr. Thompson presents an imposing array of figures to show that the railroads of the country, taken as a whole, could hardly be reproduced for the amount of their outstanding stocks and bonds, and then he goes on and shows, from these figures, that the money invested in railroad securities is only earning a very moderate rate of interest.

This would appear to settle the question, but does it? Let us inquire as to the source of these stocks and bonds.

Many of our roads were financed and built on what a noted captain of industry characterized as the "package system".

A corporation was organized with a capital of say $1 000 000 in common stock, and $1 000 000 in bonds was immediately issued. A lot of preferred stock was also issued in some instances. These were sold to investors or to a syndicate at the best price obtainable, that is, the bonds were sold, generally below par, and a bonus of stock was given with each bond subscription. This was the "package".

The road was built from the proceeds of the bonds, the remainder of the stock being retained by the promoters. We now have a road which cost not more than $1 000 000 and even less, if less than par was realized on the bonds, but which is capitalized at $2 000 000 or more in stock and bonds. We will assume that this occurred twenty or thirty years ago. The road since that time has been ably managed, and sufficient money has been saved from earnings to keep the road in good repair, and in addition make continual improvements, such as ballasting the track, replacing wooden bridges with steel and concrete structures, buying new and heavier locomotives and modern equipment of all kinds, until now the road is actually worth the amount of its capitalization. Consequently, the stockholders insist that there is no water in the capitalization, as the road could not be duplicated for any less than the amount of its outstanding securities, even if it could be done for that, on account of present higher prices. But, whose money has made this present value? Has any one put in anything since the original construction of the road, except that which the public has paid in rates? It does not need a diagram to demonstrate that one-half at least of the capitalization is water, or is, at any rate, the profit made by the stockholders who originally put in nothing.

It will be urged that the promoters were entitled to a good profit, and that they and their successors are entitled to this increment on account of their able and efficient management, and to a certain extent this is true. The fact remains, however, that if a valuation is made having regard to the original investment, some one has made quite a lot of money at the expense of the public.

This particular state of affairs, of course, did not occur in all cases, and perhaps not even in a majority of them, but it serves to show the fallacy of the position that because the present value of the roads is

Mr. Hand. equal to their capitalization, if it is, that there is no water in such securities.

When the valuation of the Interstate Commerce Commission is complete we will have a much better idea where we stand in all these matters than we do now, and the results will undoubtedly be worth much more than their cost.

Mr. Norcross. P. H. NORCROSS, M. AM. SOC. C. E. (by letter).—Mr. Wilgus has presented a paper which deserves not only the approval of the Engineering Profession, but also that of the legal and commercial world. He has analyzed the fundamental principles of valuation of public utilities and presented them in such a clear-cut and concise form that he has left little opportunity for discussion. The writer feels safe in asserting that this paper will become a notable addition to that rapidly growing branch of engineering literature, "Valuation of Public Utilities".

The writer believes that "valuation" or "value" is one of the most misunderstood words in the English language. It is like a sharp-edged tool, which, when used by a skilled artisan, will perform the functions intended, but, when handled by one not competent and experienced, will become a menace and a danger.

The Matter of Value.—The work to be performed by the specially appointed Valuation Committee of the Interstate Commerce Commission, involves the determination of the value of the most important and largest branch of public utilities. A railroad is necessarily property of such a character that it has not what is generally known as "market value". It is not like a commodity which is frequently bought and sold on the market, the large number of sales thus forming a basis for determining that which is usually termed "market price". This condition does not usually exist with public utilities. The circumstances may be such that the utility being considered is not salable. Even if such a condition existed, it would not affect the question of "value".

"First and foremost the determination of the value of a public utility is a matter of judgment, and the accuracy of the determination depends upon the integrity, fairness, technical training, and ability, character, experience, and sound good sense of the man (or men) who makes it."*

The writer agrees that the practice of estimating "Cost of Reproduction New" is a prominent factor in determining the value of property. This principle has been adopted in the commercial world as a proper method for determining value. It is most important to have a clear and comprehensive understanding of this method, otherwise a correct conclusion cannot be reached.

* "In the Matter of Condemnation Proceedings Before a Condemnation Court," Des Moines, Iowa, *vs.* Des Moines Water Company; Brief by Guernsey, Parker and Miller.

Mr. Norcross.

The appraiser must place himself in the position of some one who is about to build a railroad, and must go through all the different processes from its conception to its completion and operation, so as to arrive at the proper value of the physical property, revenue, and other characteristics, which are all elements of value in the cost of the complete system.

One of the most important things involved in the estimate of the cost of reproduction is an inventory of the property to be reproduced, and the preparation of this inventory is naturally one of the most arduous tasks that will be undertaken by the Valuation Committee. This inventory, when completed, is the basis for estimate in the cost of reproducing the physical property, and should be made by honest, competent, intelligent, and experienced engineers.

As to the adoption of unit prices, these should be based on actual experience, covering construction work of similar nature under similar conditions, and should not be predicated on more arbitrary estimates.

What is to be ascertained is the value of the property to-day. Any attempt to estimate the cost of reproduction under conditions that existed, or might have existed, at the time the property was constructed, would most probably result in an estimate so inaccurate that it would not serve the purpose for which it was made, as it might be more or less than its present value.

As to whether depreciation should be taken into account depends entirely on the purpose for which the valuation is made. If we are seeking present value, then depreciation is material. If cost is what we are seeking, depreciation is absolutely immaterial. The very reason for taking depreciation into account to ascertain present value, however, requires that appreciation be given the same importance. The writer is aware that there are authorities who take an opposite view, but nevertheless, in his opinion, the foregoing is the correct method of determining present value.

At the present time, the work of the Valuation Committee is only in its incipiency and such a paper as Mr. Wilgus has written should be of inestimable value to the members of the Interstate Commerce Commission.

Mr. Harte.

Charles Rufus Harte, M. Am. Soc. C. E. (by letter).—This paper is a masterly consideration of the subject, and though specifically applied to railroad valuation is, in fundamental principles and in much detail, equally pertinent to other cases.

The author's professional standing and experience give great weight to his conclusions, but, as is almost necessarily the case in the earlier consideration of any question of moment, there are many who, viewing the situation from other angles, will find points of difference, particu-

Mr. Harte. larly in the matter of the treatment of depreciation, this independently of the series of consistent decisions by the U. S. Supreme Court.

Primarily, although many of the elements entering into it are difficult of exact determination, a valuation is the establishment of a fact, and as such it should vary only as the personal equations of the experts employed in different determinations are reflected in the variables; its intended use should have no consideration in establishing fundamental values, although it may determine what of these values shall make up the total to be obtained.

As to the method to be followed in determining this fact of the value, the "Sale Value" as defined by the author might more properly be termed the "Scrap Value," disregarding as it does not only the complex and moot factors arising from a "going" concern, but also the obvious fact that the assemblage of the parts into a complete but "dead" system is affected only at a very considerable cost over and above the price of the constituent materials as such.

Only those who have attempted the task can fully realize the utter hopelessness, in at least many instances, of determining with accuracy the "Original Cost to Date;" nor is the result obtained under the most favorable circumstances of necessity a correct valuation. "High Finance" methods on the one hand, whereby the directors of a railroad pay to themselves, as directors of a "Construction Company," contract prices far in excess of normal, or abnormally low prices on the other hand, to say nothing of items omitted from the record, will give an "Original Cost" which was not a true value at the time of construction, and may well be much more inaccurate at a later date, while appreciation or depreciation, both of which should have full consideration, are entirely ignored. The U. S. Supreme Court, in the Minnesota Rate Case decision, simply adds one more to the consistent chain of precedents when it says:

"It is clear that in ascertaining the present value we are not limited to the consideration of the amount of the actual investment. If that has been reckless or improvident, losses may be sustained which the community does not underwrite. As the company may not be protected in its actual investment if the value of its property be plainly less, so the making of a just return for the use of the property involves the recognition of its fair value if it be more than cost."

As to whether the reproduction cost shall be figured "New", or "Less depreciation" there can be but one answer: "Both"; for both are essential to a proper understanding of conditions. The author argues for "Reproduction new", but it would seem that in doing so he takes too much for granted. It is true, of course, that a sinking fund should amortize those wastes of substance which in practice are replaced only at considerable intervals, and after the loss has reached a critical value; and that regular maintenance should care

for every change that can be met as it occurs, but these are facts to be determined, and, except as it can be shown that the decrease in physical value due to such wastes actually, in the specific case considered, has been compensated by funding provisions, and such fund is made a part of the valuation, the physical value total must reflect the depreciation. Mr. Harte.

In the case of the coal pocket cited by the author, it will hardly be contended that, in its all but broken-down condition, the pocket has the same physical value as after the subsequent complete rebuilding —nor can there be doubt that, with a properly maintained and applied amortization fund, the total value of the railroad as a whole is the same at each time.

"If, however, a company fails to perform this plain duty" (to make provision out of earnings for replacement of waste) "and to exact sufficient returns to keep the investment unimpaired, whether this is the result of unwarranted dividends upon over issues of securities or omission to exact proper prices for the output, the fault is its own. When, therefore, a public regulation of its prices comes under question, the true value of the property then employed for the purpose of earning a return cannot be enhanced by a consideration of the errors in management which have been committed in the past."*

"It must be remembered that we are concerned with a charge of confiscation of property by the denial of a fair return for its use; and to determine the truth of the charge there is sought to be ascertained the present value of the property. The realization of the benefits of property must always depend in a large degree on the ability and sagacity of those who employ it, but the appraisement is an instrument of public service, as property, not of the skill of the users. And when particular physical items are estimated as worth so much new if in fact they be depreciated, this amount should be found and allowed for. If this is not done the physical valuation is manifestly incomplete. And it must be regarded as incomplete in this case."†

It is interesting to note that the March 1st, 1913, amendment to the "Act to Regulate Commerce" of February 4th, 1887, providing for the valuation of property of carriers subject to the act, requires the determination of all the above methods except the sale value, calling for "the original cost to date, the cost of reproduction new, the cost of reproduction less depreciation, and an analysis of the methods by which these several costs are obtained and the reason for their differences, if any".

Considering the details treated, the recognition of the initiatory and developmental expenses as legitimate elements of value will be assented to by all familiar with the facts, but detailed analysis of actual expenditures will undoubtedly be required to establish the fact on a legal basis.

* Knoxville *vs.* Knoxville Water Co., 212 U. S., 1, 10.

† Minnesota Rate Case decision.

Mr. Harte.

The point of many contingent items, difficult to foresee in the original construction or to recognize in an appraisal, is well taken. Quicksand, covered bogs, swelling clays, shattered ledges in fragments too large to handle with ordinary equipment and too small to blast except at high cost—the list could be carried out almost indefinitely—all are predetermined only by much more careful and expensive investigation than is practicable in the majority of cases, and are still more obscure after the work. The author notes the case of rock in a cut masked by earth washing over it, but here some clew may be had to earlier conditions. In the case of drained sections, however, quicksand or bog conditions which caused exceedingly heavy expense will often be completely removed after some time by the lowering of the water-table.

In the case of fills, it would seem difficult to sustain successfully an appreciation equivalent to the cost of compacting by puddling and rolling, as such a condition of density is not essential. There is, too, room for difference of opinion as to the extent to which it is proper to include costs of cleaning out cuts after operation has begun. With traffic of any moment, such work is extremely expensive, and though there can be no question as to the propriety of considering as a part of the value of the property the fair price of every yard removed, there is good argument against the inclusion of the excess due to work under regular operation, on the ground that such necessity arises from improper first construction.

The use, in determining interest, of "consistent" progress, is at least conservative. The condition of one or two critical points where unforeseen contingencies of one sort or another have delayed operation of the system until long after everything else is in readiness, is not at all unusual; and, under the regulations of to-day, a system ready for operation may be held idle for a considerable time pending the necessary authority. A case to the point is that of the Hampden Railroad, in Massachusetts, which, ready for operation on June 1st, 1913, after an expenditure said to have been practically \$4 500 000, at this writing, 6 months later, is still awaiting approval of its financing.

In the matter of land values, no one with any knowledge whatsoever of the facts can sustain the contention that a strip of land for right of way does not cost considerably more per unit of area than adjoining land purchased in normal parcels. Interference with the use of the portion cut off, where the property is divided; reduction of the remainder to undesirable or uneconomic size—these are takings of property rights in the portion left as real as if the land itself was taken and must be—and in condemnation awards invariably are—given proper compensation. No less effective in enhancing legitimate values is the advantage of the situation often taken by shrewd owners. It is true that a railroad has the right of eminent domain, but the exercise

of this right adds to the award the cost of the proceedings, and the delay—a case in Connecticut, begun in 1908, was not settled until the spring of 1913—often involves such expense in interest on the suspended construction as to make payment of an outrageous price much less expensive than to wait for the more reasonable Court award. Mr. Harte.

In the Minnesota Rate Case, the Court, it is true, refused to allow "the so-called railway value of the property", but that this was due to faulty presentation of the case is obvious in the opinion:

"But aside from this, it is impossible to assume, in making a judicial finding of what it would cost to acquire the property, that the company would be compelled to pay more than its fair market value."

And again:

"The Company would certainly have no grounds of complaint if it were allowed a value for these lands equal to the fair average market value of similar land in the vicinity, without additions by the use of multipliers or other use, to cover hypothetical outlays."

These quotations clearly show the failure, on the part of the railroads concerned, to develop legally the fact that "the fair average market value of similar land in the vicinity" is the value of land in right-of-way shape, and not that of land in normal parcels.

In this connection the engineer engaged in appraisal should bear strongly in mind what is too often forgotten or overlooked, not only by laymen, but by lawyers as well: The expression, "Blind Justice" as applied to the Courts is no mere figure of speech, but is a cold, hard fact. The lower Courts have very little leeway in considering any facts in a case, other than those legally presented, "judicial notice" relieving from proof only such facts as are of common knowledge beyond any question; and the higher Courts are bound absolutely to the records of the lower Court from which came the case.

The facts, therefore, in an appraisal—and for that matter in any engineering report—should be set out so that they not only can be understood, but that they cannot be misunderstood.

It has been argued that land occupied by a railroad is not entitled to any appreciation over its first cost, on the ground that such an appreciation is due to the railroad, and that if the latter were destroyed or removed the appreciation would be lost. This, however, is a question of fact. To a growing community with insufficient transportation facilities a railroad will almost invariably bring increased general values; but, except as it creates unusual manufacturing or other commercial facilities, it will depreciate its immediate vicinity; to an older settlement, having fair service, such railway improvements as additional trackage, increased terminal facilities, and the like, rarely add to general values, and almost invariably depreciate the territory affected.

Mr. Harte. In fact, the entire question of land and all other values harks back to the proposition that an appraisal is a statement of fact, depending on many variables; given an outline of the conditions, the use of averages will give a fair approximation to the total; but that appraisal which is to stand the test of the Courts—and that is the final test—must be based on, and must show beyond question and in detail, the elements of which the total is made.

Mr. Green. F. W. Green, Assoc. Am. Soc. C. E. (by letter).—As the subject of valuation occupies such a prominent place in public thought at the present time, a full discussion by the members of this Society will be most opportune. That correct principles may be expounded eventually, and erroneous conceptions convincingly exposed, is a consummation "devoutly to be wished."

On pages 307 and 308 occurs the statement:

"A railroad, or, in fact, any other public service corporation, does not occupy the same position, relatively to the public, or to the State, as a private citizen. A corporation has no being and no existence until created by the granting of its charter by the State, which charter defines in just what direction the corporation is to exercise its functions. A railroad corporation is thus granted rights, privileges, and immunities by the State, which are not accorded to the individual. It is a fundamental axiom of both law and economics that something cannot be obtained for nothing, and thus, in return for these grants, the railroad must give up some things to the State, which the latter cannot demand from the individual."

This reflects the popular attitude, but is it correct? It is said that the corporation originated in the days of the Roman Republic, when a vast undertaking, requiring a long period of time for its completion, necessitated the creation of "a body without decay, a mind without decline." The production of wealth flows from the application of capital and labor to the development of natural resources. Public wealth is measured by the magnitude of private wealth. The development of natural resources, therefore, must inure to the public benefit. Would our country be as well developed and as prosperous to-day if the organization of corporations had never been permitted? Could our present development have been attained by individuals or by co-partnerships?

A nation may be likened to a corporation—its life is derived from its Constitution. Does not such a nation gain "something for nothing" when it assumes its Governmental functions? But, we say, the Government, in return for what it receives, gives protection to its citizens, and therefore reciprocal benefits are enjoyed. How is it in the case of a railroad corporation? It derives its life from its charter. No sane Government would give life to a corporation unless it expected public benefits to flow therefrom. The "rights, privileges, and immunities" granted by the State to the corporation presumably refer to

the right of eminent domain. Does the State confer this right from motives of prodigal generosity, or *pro bono publico?* Does the corporation, in the enjoyment of it, get something for nothing, or does it pay a handsome price for condemned right of way? Any one with experience in condemnation suits knows the answer. Mr. Green.

Elsewhere in the discussions appear references to the acquirement of rights of way by public or private donation, and intimations that the people should now turn "Indian givers" by renouncing the acts of their forefathers. If such acquirements had been obtained by fraud, the ethics of such a procedure could hardly with justice be questioned; but, when these ancient transactions occurred, were the railway corporations the only beneficiaries? Does the construction of a railway through an undeveloped country produce any increment in land values? Was the value of the land acquired for right of way, before the construction of the railway, more than a small fraction of the increase in land values enjoyed by the owners after its construction? Therefore, was the donation of the right of way a betrayal of the rights of posterity, or a shrewd business move? If, by the construction of a railway through an undeveloped country, the corporation is the means of doubling land values, the increment accruing to the landowners being entirely providential, is society treating the corporation with equity and fairness when it says: "Thou canst take the chances of going broke, but if thy venture shall profit thee, thou shalt enjoy only the legal rate of interest"?

The writer believes in the existence of rational grounds for making the rate a function of the valuation in such utilities as gas companies, telephone companies, etc., but has the temerity, here and now, to express the opinion that the application of this theory to railway rates is utterly fatuous, and predicts that, in so far as materially altering present rate conditions is concerned, the Federal Valuation of Railways will prove to be "much ado about nothing."

It would be futile, however, even to attempt to stem the tide of public clamor for valuation. One who has had no experience in physical valuation can have no adequate conception of the obstacles encountered in such work. The writer's limited experience in this class of work has already taught him the necessity of depending on the actual engineering and accounting records, whenever they are available, rather than mere conjectures as to what the cost ought to be. For example, a certain passenger station would be appraised by probably nine out of ten experienced and expert engineers at about \$75 000, original cost. An examination of the original plans and accounting data, however, reveals extraordinary foundation difficulties, the conquest of which cost nearly \$20 000 more than the amount that would ordinarily be estimated. Again, a certain right of way was obtained through condemnation proceedings at a cost somewhat in excess of \$100 per

Mr. Green.

acre. This was the cost to the corporation, although the actual value of the land at that time was not more than $35 or $40 per acre. Would an appraisal which failed to take such matters into consideration do full justice to the corporation?

Again, it is argued that original cost should constitute present value for valuation purposes. Is it equitable to deny to corporations the increment which is freely conceded to others?

Referring to the proposal to deduct depreciation from cost of plant: As pointed out so ably by Mr. Humphreys, plant maintenance must be provided for out of earnings, and it would be contrary to reason to make it a charge against capital. On the other hand, the very purpose of depreciation reserves is to maintain unimpaired the capital originally invested—to keep the capital alive, as it were. As long as such reserves are provided, and original capital is not depleted, no deductions should be made from original cost of plant.

For valuation purposes, a railway should not be considered merely as a collection of various items, the value of which could be determined from an inventory. It should be treated as a living organism, so to speak. There is a vast difference, for example, between the summation of values of the constituent parts of a mule, when he is dead, as compared with his live value as a "going concern."

Mr. McCormick.

R. S. McCormick, M. Am. Soc. C. E. (by letter).—In undertaking the task of placing a valuation on railroad property, the Interstate Commerce Commission has been granted large powers, and the results of this valuation will be vital to the railway companies of the United States.

The figures furnished by the Commission will certainly be utilized for the purpose of fixing rates, or at least, in adjusting rates, and it is quite probable they will also be used in fixing the compensation to be paid in the purchase of railroad property outright by the Government.

The very able and complete manner in which Mr. Wilgus has presented this subject leaves little room for criticism by members of the Society familiar with railroad conditions. That the method of valuing by "Cost of Reproduction" is eminently fair can hardly be denied, and hence, seemingly, the only thing to discuss is the question of depreciation. Volumes have been written by a multitude of authors on this subject; the Courts have decided that "depreciation" must be considered in valuing public utilities, and, in view of the recent decision in the Minnesota Rate Case, the method of "Cost of Reproduction" with depreciation considered will probably be used in whole or in part in valuing the roads.

Just what depreciation is and how it should be applied in railroad valuation is a big question. Science teaches the indestructibility of matter, and we are all agreed that, once created, matter cannot be destroyed. We all know, however, that it can, and does, change very

materially. The component parts of a railroad company's property are made up of materials and things which are in continual process of change in form. They have a certain value in one form and perhaps another value in another form, until, as time goes on, they are of no value for their original purpose, and are dispensed with, or, as we say, renewed. Between the minute they were first used and the minute they will be discarded, some change in shape, form, appearance, or general utility has been going on, but, until actually discarded, their usefulness is not impaired as affecting other items; it only takes a lower standard. Mr. McCormick.

It is agreed by all that railroad property, by the very nature of its use, must be considered in a different light from ordinary private property. This is reasonable because of the large sums invested in such property, and because, speaking generally, it can only be used for one purpose, and for any other would be valueless, or, at least, of greatly lessened value; hence, it seems clear that its true value is made up of its value for this purpose and no other.

This value can be ascertained, in all cases, by the method so well outlined by Mr. Wilgus, namely, the Cost of Reproduction, provided intelligent men are guided by right principles so as to know when they have reproduced the property.

It is not true that making new every component part or unit of a railroad is reproducing that railroad. All will recognize this fact. Neither does the writer consider it fair treatment to such property that actual physical depreciation based on age be written off against each item or unit, when, as a whole, the item or unit, as classified, say, under the Interstate Commerce Commission Classification, is 100% efficient when joined with other items.

Therefore, a fair and equitable valuation must be based on the following principles:

(*a*). A standard should be fixed for the items under the Interstate Commerce Commission Classification, as far as these accounts cover measurable items, and all items included in these accounts should be given a marking, up to 100. This marking, however, is not to be based on the age of the item, but on its condition for the purpose for which it is utilized. The standard set as 100 should be a brief and explicit statement, applicable to each account, and in this way—from information as to condition, etc., taken in the field—the value of the item as a component part of the whole, rather than as a separate entity, can be determined. This arrangement should be followed for each mile of main track, for each 5- or 10-mile section, or otherwise, as seems most convenient. To show how this would work out: Account 4, "Grading," through Account 16, "Interlocking and Other Signal Apparatus", includes all accounts and all items covering a section of rail-

Mr. McCormick.

road track. For each of these accounts, the writer would define a standard and give that standard a rating of 100. "Grading", for instance, Account 4, would have a standard equal to a first-class, workman-like piece of construction, full width in all cuttings and embankments, well-shaped slopes, etc., etc., in fact, such a standard as any competent railroad construction engineer could easily define, keeping in mind the requirement mentioned under (*b*). All measurements of quantities are then made from the profile or in the field. If the work as performed meets the requirements of the standard, values are fixed at full prices to reproduce. Compacted embankments, seeded slopes, and other betterments of age, but on which no actual money was expended, will receive no advantage over newer work, except in the matter of adjusting quantities to cover same.

Now take Account 6, "Bridges, Trestles, and Culverts": A standard, covering the classification written as guided by (*b*), will be prepared. All structures will be measured, and their value estimated or fixed by data at hand. If the structure comes up to the standard, it is valued at renewal prices with no depreciation, though it may not be a new structure. The standard fixed for this classification must have sole reference to its condition and usefulness for the purpose, and not to its age. An honest engineer can grade such structures and ascertain their actual value by this method better than in any other way, and when the standard is once fixed, the proper value can be readily determined. Again, take Account 7, "Ties," or Account 8, "Rails": There will be ties in the track, and rails, also, which are not new. Could they be reproduced to answer their present purposes by any other means than by buying new ones? If not, then the standard should specify under this account that only new ties and new rails can be marked 100. Materials of this class should be valued at renewal prices and marked 100 if up to the standard in other respects.

Again, take Accounts 37 to 42, inclusive, "Equipment": Fixing a standard, based on condition, will obviate the large errors made in valuing such property by writing off depreciation based on life. Does any one doubt the efficiency and value as property, say, of a locomotive in good repair, which has not been worked beyond its capacity and is in every respect fitted to do its work? To a railroad company, is not such a machine fully as valuable as a new engine? In many cases it is more valuable. A standard, therefore, fitted to define what condition is entitled to 100, regardless of age alone, will give an equitable value, based on cost to reproduce this article as a component part of the whole. Every other account can be treated in the same way.

(*b*). In establishing the standard, locality or district should govern; that is to say, the same standard must not be applied to all sections of the country. The sections or zones should be first defined and a standard established for each.

Mr. McCormick.

(*c*). Land values should be standardized over each section or district, "Right of Way and Station Grounds," Account 2, and "Real Estate," Account 3, being separated. For real estate, values must be given as determined by adjoining property; for right of way and station grounds, a standard rating for railroad lands, based on values known to exist in that section of the country. The market value of adjoining lands may be used to assist in determining this standard rating.

By following these principles, a true valuation can be obtained. By this method the values placed on all roads will include additions and betterments which have been charged to earnings and are not represented by bonded indebtedness; hence, all physical property will be valued at its real worth, and if these additions and betterments have been made wisely, they will be represented by actual property value; if, on the other hand, they have been made extravagantly or wastefully, they will not be represented, and will not be accounted for. In the one case property is in evidence, in the other, investment; and we are all aware as to which can be protected legally.

As for values not represented by physical property, that is, promotion costs, interest charges, law expenses, etc., which go to swell the actual cost of a railroad property: these, of course, cannot be valued like physical property, but, under the proposed plan of referring all data to standards, they can be given consideration, as follows:

All items of this nature found charged up to Classifications 43 to 48, inclusive, should be totaled with Classification 1, "Engineering," where these records are available. After the full physical valuation is made for the entire system, compare the actual determined values with this total. After obtaining a full history of the system, leave it to a commission to fix a proper sum to add to this valuation to ascertain the true total valuation. Surely engineers know there is a definite relationship between the value of a work produced and the costs noted above to produce it. To create infers the utilization of the services of a creator. Certain legitimate charges for such services must, and no doubt will, be permitted in the total valuation. To reproduce new any one of the railway systems of the United States involves incorporating into the value of the inanimate objects composing the property, the cost of joining them together to make of the whole a living, running organism, and such costs are legitimately represented under "Engineering" and items under "General Expenditures" in the classification.

On American roads, during late years, large expenditures have been made which are of great benefit to the public, but earn nothing for the roads themselves. All such additions and improvements will tend to swell the total value for rate-making purposes.

After all, however, rates can hardly be based on values defined for each individual company—at least, not through rates. The only ad-

Mr. McCormick. justment, therefore, as stated by Mr. Lavis, is a blanket one for that section, and for that reason the writer considers his plan of a standard for each section the only practicable way of settling the question of depreciation and ascertaining how such a valuation can be utilized in rate adjustments or rate-making. Through rates will always be regulated by agreement between roads in the territory, under private ownership, as done at one time by competition. The powers of the commission can hardly go farther than to fix a maximum beyond which the roads cannot go. The road best equipped physically, due to low gradients, etc., and best managed will get the business or the lion's share, as at present. The rate covering a district or section can be regulated, and this rate should be based on a fair return on the true value of the property.

Mr. Corner. CHARLES CORNER, M. AM. SOC. C. E. (by letter).—The writer has read with much interest Mr. Wilgus' paper, together with the discussions thereon, and notes that his conclusion seems to be almost that of the pioneers in this important subject.

The matter was originated by the late Governor Hogg, of Texas, under whom the Texas Stock and Bond Law and Railroad Commission Act were passed, and administered by the first chairman, the Hon. J. H. Reagan, in the Nineties. It will be remembered that Senator Reagan was the Father of the Interstate Commerce Commission as well.

Subsequent work on this subject, throughout the United States, has arisen from the initiative of these men, and, as Engineer of the Railway Commission of Texas, from 1893 to 1898, the writer did a great deal of work on the valuation of the Texas Lines, which, up to that time, had not been contested by the companies interested. Communications by the writer on this subject were forwarded to *Engineering News*,* and are included in the "Bibliography on Valuation of Public Utilities."†

This "Bibliography" is of the utmost interest and value, but the writer does not think the Annual Reports of the Texas Railways are included in it. Certainly the reports for the dates referred to throw the greatest light on the beginnings of the movement, the credit of which belongs so largely to Governor Hogg and the late Judge Reagan. After the Texas policy was inaugurated, the latter was in direct communication with several other Southern States, which, thereafter, adopted similar policies and views.

The writer thinks that this is a matter which should be put on record, because, at the time, their action excited the greatest discussion and opposition. Their general views have been amply justified; the position which they took at that time was that of far-seeing statesmen.

* March 7th, 1895, and April 23d, 1896.

† *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 1280.

WILLIAM J. WILGUS, M. AM. SOC. C. E. (by letter).—The writer has a deep sense of obligation to those who have kindly participated in the discussion of his paper, as many of his own ideas have been clarified by the additional light that thereby has been cast on the subject. Mr. Crehore has generously credited him with an absence of desire "to complicate valuation work, and to seek out every possible pretext for making the assets, both tangible and intangible, look as big as possible"; and to this the writer feels that he may conscientiously reply that, in preparing this paper, he has had absolutely no other purpose in mind than an attempt to place his own views where they may be freely criticized by his fellow members and others, all in the interest of straight thinking. Mr. Wilgus.

Unlike Mr. Crehore, the writer does not believe that the *bona fide* stockholder as a class "gets something for nothing," nor that the stockholder is an "incumbrance." The fact that a small group of financiers may have abused the confidence and trust of innumerable innocent, if too confiding, investors of small means, seems in fairness to be no reason why experienced engineers should not attempt to formulate principles for guiding a valuation of property capitalized at some $20 000 000 000, for purposes that in the end may crystallize into a move for Government ownership.

Surely, Mr. Crehore would not ask that the subject shall be approached other than dispassionately in the hope that correct principles of valuation may be adduced, and that the facts as to any improper past return to the investor through interest and dividends and reinvested income, shall be then studied and a fair decision reached in the case of each road. Broad generalizations as to inordinate profits to investors in public utilities can hardly fail to do an injustice to a large proportion of the public that in good faith has invested amounts, great and small, in enterprises from which the entire public has reaped the enormous benefits referred to by Mr. Churchill and Mr. Green.

There are, no doubt, cases like those referred to by Mr. Hand where the public, through the rates, has paid an excessive return; but each instance requires detailed analysis to ascertain whether or not the annual distribution of cash in dividends and interest, plus the surplus reinvested in the property, really constitutes an inordinate reward to the investor. Many roads have been built up by putting back into the property moderate earnings that otherwise, with propriety, might have been distributed in cash to the stockholders, a praiseworthy proceeding that merits warm approval rather than condemnation.

In this connection it will be well to bear in mind that an approval of the theory that reinvested surplus and increases in land values are to be viewed as additional income to the stockholder, necessarily carries with it the proposition that such additional income is entitled to a return. This admitted, the accrued re-investments and other increases

Mr. Wilgus. in value constitute a sinking fund, made up of equal annual payments compounded at the rate of interest to which the service is entitled, for the full period of accumulation. For instance, an increase in 60 years of $100 000 000 in the value of a property, due to the increments mentioned, is equivalent to $187 572 annually, compounded at 6 per cent. This annual sum, added to the average annual dividends and interest actually disbursed in a given time, will produce the figure that should be taken as the total past annual return to the owners of the property in both cash and "kind". The point that the increment, if interpreted as income in an analysis of past results, is entitled to a return, has been missed by many. A similar error has been made by those who would count the increment as an additional return to the owner, and at the same time brand it as a depreciation reserve.

Several, notably Mr. Eaton, have expressed regret that the broader social problems of the day have not been touched on in the paper. The writer has felt that the subject of the valuation of railroads should be treated concretely, with a definite end in sight—the logical method of arriving at physical values—and that the manner in which the result should be used is a separate matter for independent discussion. The treatment of the so-called unearned increment, including donations from the community and reinvested surplus out of questionably excessive earnings of earlier years, is a question on which few as yet agree, and until the views of the majority are expressed through legislation that will affect all kinds of property alike, it does not seem equitable that one class of investment should now be selected for retro-active action. The correct course would appear to lie in making an appraisal that will stand the acid test of analysis, and then from a study of the past condition of each property make such disposition of the unearned increment as will be in consonance with a similar attitude toward other classes of property.

Mr. Gillette speaks of the supplanting of the competitive theory of railroad ownership with the agency theory, under which the public utility continues to be financed by the owner, but is indirectly managed by the Government through public regulation. The writer believes that this dual and contradictory relationship can be but temporary, the next logical step being nationalization. The enforcement, by the Interstate Commerce Commission and similar bodies, of expenditures for improvements on railroads will make obligatory the raising of new money by the owners through the sale of bonds or other form of prior obligations, and this in time will materially reduce or wipe out the stockholder's equity. If this final step to Government ownership is inevitable, it would seem better for the stockholder to encourage the transfer while he still has something to sell, that is, before his equity has been legally confiscated.

Mr. Wilgus.

There is a widespread feeling, evidenced by the remarks of Messrs. Eaton and Hand, that there should be but one measure of value. The laws of some States fix values for taxation purposes at "scrap-value," and the results of physical valuations on that basis, as in the instance of New Jersey, have been presented by technical journals to their readers, without cautioning them that the figures represent scrap and not reproductive values. The difference between the two may be made clear by citing the illustration of the office chair that is worth the full cost of reproduction as a part of a going concern, whereas, thrown on the market, at auction, it would bring a mere fraction of its cost.

The contributors to the discussion by no means agree on the principle that should be adopted in determining physical value. Messrs. Churchill, Gandolfo, Gillette, and Hand, see the practicability of ascertaining the original costs from the books or the amount of cash actually invested, while Messrs. Coombs, Harte, Humphreys, Ingersoll, Lavis, Molitor, Thomson, and Whinery join with the writer in considering this course to be generally impracticable. An intimate acquaintance with the records of many corporations, large and small, has demonstrated to the writer that the obtaining of correct, full, original-costs-to-date is so generally impossible that the use of that method would work wide injustice. No doubt there are some instances where book costs have been carefully kept so as accurately to reflect every item of expense chargeable to capital account, including organization, legal and administration expenses, interest during construction, and freight on construction materials, as well as all additions and betterments, with their proper burden of overhead charges; but the exceptions are so numerous that the use of a substitute method would be imperative, with the result that all roads would not be treated alike.

An objection that has been raised to the alternative for original-cost-to-date, namely cost-of-reproduction, is the inclusion therein of the increment of land values. Many contend that railroads, being public utilities, should not profit by the increment which is freely conceded to private individuals and corporations of another character. The justice of this contention is not apparent, as there are no laws in force for the guidance of investors in public utilities, which differentiate on this point between the two classes of ownership, nor is such contention upheld by the tax authorities, as forcefully pointed out by Mr. Aldrich. Arguments on this phase of the topic are well expressed by Mr. Green.

That any other than the reproductive theory will produce inconsistent results will be seen in the following case, which is illustrative of a situation that is very common.

Of three railroads running near each other through the same territory, Line A was built 30 years ago and has partial records of original-cost-to-date, Line B was constructed recently, and has com-

Mr. Wilgus. plete and accurate records of cost, and Line C, built 15 years ago, has no records of cost whatever. If the original-cost-to-date principle is adopted, the recently-built Line B, having full records, will be treated equitably; but Line A will suffer through incompleteness of data and loss of the increment of value that is contained in the estimate for Line B, and for Line C, possessing no records, the estimate cannot be determined by the adopted principle, and, therefore, must be arrived at by some other method which will be inconsistent with that used for the rival lines. On the other hand, if the reproductive principle is adopted, all roads will be treated exactly alike, and the older lines will not be placed at a disadvantage in comparison with the last built road.

Many endorse the reproductive method in theory, but in practice propose modifications that destroy the principle. The estimating of the cost of land at the same price as neighboring property, sometimes termed the basic or normal price, ignores the facts set forth by Messrs. Aldrich, Harte, Howard, and Whinery, that land for railroad purposes actually costs in excess of such price, and that the exclusion of a part or all of the overhead costs leaves out of the estimate elements which are as essential to the creation of a railroad as scaffolding in the building of structures. The omission of these items of cost is not in accord with either the original-cost-to-date theory or the reproduction method. Estimating in this manner has not the merit of following clearly any theory that will stand analysis, and is contrary to the experience of those who are familiar with the keeping of book costs or the actual building of railroads.

It has been said that reproductive estimates provide for "what it will cost to buy again land that will never be bought again, to duplicate property that will never have to be duplicated, and to build up a business that will never again have to be developed." Is not this the course that necessarily has to be followed in arriving at the value of any going concern, the earning power of which may not be used as the measure of its value? The appraiser of the physical value of a factory would base his estimates on a duplication of the plant, including the present-day reproductive value of its lands, structures, and cost of delevopment. What would be necessary in a simple instance of this kind is just as necessary in the case of a railroad.

Referring particularly to the matter of lands, and using the previously quoted illustration, Line A will be found to have paid for right of way 30 years ago, say, $600 per acre, and the modern Line B will have paid for land of precisely the same character, say, $1 500 per acre, or three times the basic value of neighboring property which in large blocks sells at an average of $500 per acre. The original-cost-to-date principle will give Line A $600 for exactly the same kind of property which, in the case of Line B, will be valued at

$1 500; and Line C, having no records, will have its land valued by some differing method. The basic method will give Line A $500 for that which cost it $600 30 years ago, Line B will be credited with $500 for that which cost it $1 500, and, again, will Line C require some alien treatment. The reproductive method would give all three roads the same price for the same character of land, would work no injustice by confiscation, and would give to the two older lines the same values that are embodied in the cost of the new rival. Mr. Wilgus.

In the Minnesota Rate Case, the United States Supreme Court ruled in favor of the basic-price method, but the comments of the Court on the testimony of the chief witness on land values give hope that a new presentation of the subject will lead to a future modification in this regard. As is well stated by Mr. Harte, the railroads in the case mentioned failed to bring out "the fact that 'the fair average market value of similar land in the vicinity' is the value of land in right-of-way shape, and not that of land in normal parcels."

Many instances may be cited where railroads in recent years, under Court decrees in condemnation cases as well as by private purchase, have paid several times the amounts that would result from the use of the basic or neighboring-value method; and it is evident that the use of the method prescribed in the Minnesota Rate Case would work confiscation to many roads and entirely discourage the creation of new ones.

Mr. Crehore's proposal that all owners of property shall fix the value of their holdings as a basis for taxation and sale has its attractive side, but the benefits which he anticipates in the acquiring of lands for public utilities at minimum prices, would hardly materialize, in view of the question of severance and other damages that would still remain for settlement. That the property owner voluntarily should place a higher valuation on the part of his holdings through which he might imagine a railroad would be likely to pass, is crediting him with a degree of prophetic vision and engineering skill that is unpossessed by the average man; and the payment of excess taxes on such a product of his imagination would be to him a costly venture from which the chance of repayment would be, to say the least, exceedingly remote.

The question of how to handle depreciation has developed two widely divergent views. Messrs. Eaton, Gillette, Humphreys, Ingersoll, Lavis, Molitor, Taylor, and Waitt, and, to a limited degree, Messrs. Green, Harte, and Whinery, believe with the writer that no deduction therefor should be made from the cost of reproduction new, and Messrs. Brinkley, Coombs, Crehore, Gandolfo, Nicolaysen, and Willoughby think otherwise.

It is possible that this difference of opinion is to a large extent due to a failure by each side to grasp the reasons that are guiding

Mr. Wilgus. the other; and a somewhat extended dissertation on the subject may be pardonable, illuminated as it is by the ideas brought out in the discussion.

Depreciation of railroad property, as generally understood, consists of physical retrogression, due to usage and decay, and to lessened effectiveness through obsolescence and inadequacy.

This physical retrogression calls for two classes of expenditures, namely: (1) current maintenance applied to those items which need constant attention in the ordinary upkeep of road and equipment; and (2) deferred maintenance of features that cannot be economically restored, renewed, or replaced until they reach maturity.

Sound practice, enforced by the rules of National and State regulatory commissions, requires that expenditures for both current and deferred maintenance of public utilities shall be charged to operating expenses, and not to capital. Stated differently, depreciation is declared to be an item of expense to be defrayed from the rate, and not wastage of capital to be paid for through the issue of securities.

A well-run corporation will maintain its property so that current repairs and renewals will not be allowed to fall behind, and it will also regulate its distribution of dividends to stockholders so that the combined depreciation reserves and profit and loss surplus will be not less than the accruals of deferred maintenance. In this connection it should be added that the profit and loss surplus of a railroad company is just as much a reserve fund for offsetting depreciation as if so labeled.

A failure thus to provide for these two classes of depreciation is usually the result of over-payment of dividends, and eventually brings its own punishment through the necessity of a reduction or suspension of dividends during the period of rehabilitation. This course is obligatory, as charges for rehabilitation legally can only be made through income.

It will thus be seen that stockholders who unwisely or improperly overpay themselves in dividends, are simply creating a liability that must be repaid through a lessened later return on their investment. They are not, as Mr. Crehore states, in the position of "eating their cake and having it too," for the reason that they must restore the "cake" from their own resources; they are prohibited from doing so through capital account.

If this line of reasoning is correct, the investor is entitled to a rate sufficiently large to: (*a*), defray operating expenses, including current maintenance and taxes; (*b*), provide an allowance for deferred maintenance; and (*c*), yield a fair return on the investment unimpaired by depreciation; with the understanding, however, that if the allowance for deferred maintenance is not properly conserved, its

restoration shall be effected from the investors' return which otherwise would be available for dividends. In other words, (*a*) and (*b*) are preferred obligations. Mr. Wilgus.

All this brings us back to the proposition that, under the law, physical depreciation of both classes, current and deferred, is a stockholder's liability and not a wastage of capital, and, consequently, it should not be deducted from the investment in determining a question of rates.

The railroad is a very complex organism, and the question of depreciation is correspondingly involved. A more simple illustration may assist in clarifying the subject.

The owner of a ferry-boat is admittedly entitled to a rate that will produce earnings sufficient to pay operating expenses, including all repairs, renewals, and taxes, and, say, 6% on the investment. By gouging out and restoring each spot of rot or wear as it appears, and replacing each nail or bolt the moment that it commences to chafe or rust, the boat may be maintained practically new for an indefinite period; and in that event no question would be raised as to the owner's title to the full 6% return on the unimpaired investment, as there would be no depreciation. But this method of repairing, being very costly and, therefore, adverse to public interest, it is considered better practice to confine current repairs and minor renewals to those parts which may be repaired economically from day to day, and pay at regular intervals into a deferred maintenance sinking fund an amount which, with accumulations at compound interest, will produce a sum at the end of a given period with which to restore the old boat or purchase a new one. Can it be fairly said that in the latter case the return on the investment must constantly fall from 6% on the full amount when the boat is new, to zero at the date of restoration or replacement; whereas, in the former case, so much less to the public interest, the full rate should continue without abatement? If so, it would seem far better for the investor to abstain from the common carrier field, and loan his funds in a manner that will guarantee the same rate of return without confiscation of principal.

It is admitted by many that the arguments for the non-deduction of depreciation are well taken in cases where an ample reserve has been accumulated in outside investments, as in the ferry-boat example, and in cases where the property has not been enriched through improvements paid for from income; but the claim is made that in instances of physical valuation where no such outside reserve or other assets exist, and funds for off-setting depreciation have been reinvested in the improvement of the property in the shape of additions and betterments, there is a clear inequity in permitting a return to the owner on the portion of the property so reinvested. The

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answers are: First, that the impracticability of drawing lines sharply between improvements paid for through capital and others paid for from income, makes imperative the adoption of some rule that will apply uniformly to all; second, that the methods of accounting established by the Interstate Commerce Commission prohibit the practice which would become necessary of charging repairs and renewals to capital, if items originally appraised in their depreciated condition later were raised to their restored value; third, that the treatment of past additions and betterments as the equivalent of additional income to the owner removes those items from the category of depreciation reserves; and fourth, that on the owner rests the continuing duty of replacing depreciation out of the allowed rate, making good from his own share thereof any deficiencies due to over-payment of dividends or other diversions of the part of the rate intended for current and deferred maintenance.

Many are the instances which may be quoted of a reduction or suspension of dividends from the latter cause. Railroads thus situated could not increase their rates as a means of rehabilitating their properties, and have been forced to forego a full or even partial return on the investment until the necessary repairs and renewals had been effected out of what otherwise would have been distributable to the stockholders. Certainly, neither sound financing nor the accounting rules of the Interstate Commerce Commission, would sanction the capitalizing of expenditures for rehabilitation; nor would the laws of competition or the rules of regulatory commissions permit a raise of rates to accomplish the same purpose.

Not only from the standpoint of logic does it appear that in questions of rate regulation, depreciation should not be deducted from the principal, but also, as a practical matter, this course seems to be the only one to adopt. Current maintenance fluctuates during the year so that what may be a fair estimate in one month is unfair in another month. For instance, in the fall the depreciation on ties in the railroads of the United States amounts to some \$60 000 000 less than a few months earlier in the year prior to the commencement of annual renewals. Costly structures, if estimated at a depreciated value, later will be found to have been rebuilt or replaced, with a corresponding large increase in value. To add these growths in value, through restoration of depreciation, to a previously determined capital sum, in effect would amount to a capitalization of items which, under the law, had been charged to expenses—a course that speaks for unsettled rates as well as a violation of accounting principles established by the Interstate Commerce Commission. Then, too, the annual revision of depreciation, unlike additions and betterments, cannot be recorded through book entries, but necessarily must be effected through a recurrent field inspection of the multitude of items

that enter into the construction of railroads,—a truly monumental task. There is the further point raised by Mr. Molitor, that what is deemed to be a depreciated condition for high-class traffic may be as good as new for a less exacting service. Moreover, the adoption of depreciated values, with their shortened lives, would call for the inclusion in the rate of higher percentages of depreciation than would be necessary were the same objects estimated at their cost new, and this would practically nullify the saving to the public of using depreciated values as a basis for fixing the investor's return. Mr. Wilgus.

The recent disastrous flood damage in the Middle West is a good illustration of the matter at issue. Railroad property was depreciated to the extent of many millions of dollars, all of which had to be restored out of earnings and profit and loss surplus, without an increase of capital on which rate-payers would be expected to defray the interest. Surely, it would not be equitable to impose a further burden on the stockholders by a lessened return on the depreciated investment at the very time when they are compelled to forego a part of their savings or profit in meeting their liability to restore their property to full working condition. To do so would amount to a double burden on the stockholders through a wastage of capital and reduction of earnings or surplus.

Several of the contributors to the discussion consider that no deduction should be made for the class of depreciation that is repaired through current maintenance, but that deduction should be made in cases of deferred maintenance, as in the instance of the coal pocket cited by the writer, where no sinking fund has been provided. For the reasons above given, the replacement of a structure like the coal pocket is a liability of the company, to be paid for out of earnings otherwise applicable to dividends or out of profit and loss surplus; and the avoidance of the bookkeeping detail of separately labeling a fund for each structure should not affect the principles underlying the subject.

May it not be said, therefore, that in questions affecting rates, the desirability of avoiding constant fluctuations of rates, compliance with the accounting rules established by public regulatory commissions, the avoidance of confusion and needless complications, and sound logic, all point to the correctness of the claim that the value of railroad property should be considered unimpaired by depreciation; and that on the stockholder rests the obligation of restoring depreciation, either from profit and loss surplus or "reserves", or through a lessening of the return on his investment during the period of rehabilitation.

It is true that the United States Supreme Court, in the recently decided Minnesota Rate Case, did not take this view, but may we not conclude that this was the result of an inadequate presentation

Mr. Wilgus. of the principles underlying this important question, rather than a final endorsement by the Court of the general proposition that, in matters affecting return on capital, depreciation should be deducted from the investment.

The claim has been made that if the appreciated value of right of way and real estate is allowed, a deduction for physical depreciation is entirely proper. This does not seem to be a logical contention, for the reason that the appreciation of land values, being an additional return to the investor, plainly should be credited to capital account; while depreciation is of a physical nature caused by temporary retrogression, and is a charge to expenses and not to capital account.

On the very important item of overhead charges there appears to be little difference of opinion among those discussing the paper.

Beyond question, as brought out by Mr. Gillette, the percentage for contingencies should be comparatively small if reliable data as to quantities and up-to-date costs are obtainable from the records; but where estimates are necessarily made without the benefit of such precise knowledge, and the adopted unit prices are not inflated, a sufficiently liberal allowance for contingencies should be made, precisely as would be done by the experienced engineer in preparing preliminary estimates for any project. It has always seemed to the writer that the item of contingencies should be provided for through a percentage that will be open to inspection and discussion, rather than through an arbitrary increase of unit prices where it would be more or less concealed.

Mr. Gandolfo considers that no provision should be made for the educational stage during which construction gradually draws to a close, and traffic is built up from nothing to the full volume on which the income that may be in question is earned. With an object so simple as an automobile, the dealer devotes his time gratis to educate a customer and to the repair and replacement of any defective or imperfect parts, before the transaction is closed; and the cost of labor and materials for thus breaking in the car and educating the user is contained in the price. So, too, with a railroad, must certain expenses be provided for in the estimated cost of reproduction, for breaking in the plant, and educating the forces which are to supervise the moving of the existing volume of traffic. A railroad's operating organism does not spring, Minerva-like, into being, fully equipped and trained for a complicated service; it must pass gradually from the period of active construction to the culmination in growth of the going concern; and, as Mr. Lavis points out, it is by a suitable allowance for interest charges that this going value may be measured.

In the organization of the valuation corps largely rests the success or failure of the outcome of a physical valuation. Beyond question, the qualities mentioned by Mr. Gillette are desirable, and, in most instances, essential; but the one feature which he omits, experience, would seem to be the prime need, if the results are to stand. This is particularly mentioned by Mr. Waitt and Mr. Norcross. The chiefs of field parties certainly should have the very qualities that Mr. Brinkley would not credit to them, capacity and experience for observing and recording the condition of the property. Mr. Wilgus.

As stated by Mr. Molitor, a low-grade line through a populous country has a distinct advantage over a competitive high-grade line through a sparsely settled region; but with net earnings, in which the rate at issue is a factor, eliminated as a measure of such advantage, the selection of some other yardstick of intangible values is a problem on which no light has yet been shed. The writer has purposely abstained from suggesting a measure for such intangible values as traffic productivity and operating effectiveness, because he has been unable as yet to define in his own mind just how they may be estimated, other than for comparative purposes under known local conditions.

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TRANSACTIONS

Paper No. 1288

DERIVATION OF RUN-OFF FROM RAINFALL DATA.

By Joel D. Justin, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. L. J. Le Conte, R. B. H. Begg, R. G. Clifford, Robert E. Horton, J. William Link, J. K. Finch, and Joel D. Justin.

In the study of a drainage basin which is to be utilized for water supply or water power, it frequently happens that there is a dearth or utter lack of run-off data, though more or less precipitation data are almost always available.

It was with the desire of finding out if it were not possible to develop some rational method of deriving run-off from rainfall data on various water-sheds, that the present study was undertaken.

The quantity of rainfall appearing as run-off on any water-shed is governed by many conditions, chief among which are character of vegetation, extent of forest covering, prevailing winds, relative humidity of atmosphere, barometric pressure, percentage of water surface, geology of basin, slope, and mean annual temperature.

Before investigating the manner in which the relations between rainfall and run-off differ on various water-sheds, the writer will examine into the manner in which they vary from year to year on any one water-shed.

The late George W. Rafter, M. Am. Soc. C. E., showed in his paper, "The Relation of Rainfall to Run-off,"* that the relation of rainfall

* Geological Survey, Water Supply Paper No. 80.

to run-off on a water-shed could often be expressed as an exponential equation. For the Upper Hudson he gives the two equations, $P^2 = 84.5\ R$ and $P^{1.77} = 34.3\ R$; P being the annual precipitation and R the annual run-off, in inches, on the water-shed. He does not, however, suggest any constant value for the exponent of P or R for other water-sheds.

After plotting many rainfall and run-off data, the writer became convinced that the relations between rainfall and run-off on almost every water-shed could be expressed by a logarithmic equation, of the form, $C = K\ R^n$, in which C = annual run-off, R = annual rainfall, K is an abstract number, constant for any one water-shed, and n is an exponent constant for the water-shed.

In solving the equations it was found that n always came out nearly equal to 2; hence it was chosen as a constant for all water-sheds and always equal to 2.

The writer then plotted, on logarithmic cross-section paper, using annual run-off in inches as ordinates and annual rainfall in inches as abscissas, a considerable portion of the available reliable data. He found that on all these water-sheds the relation may be well expressed by the formula:

$$C = KR^2,$$

in which C is the annual run-off, in inches; R is the annual rainfall, in inches; and K is a constant which is different for each water-shed, and has a value depending on those conditions which make the relations between rainfall and run-off on one water-shed different from those on another.

Most hydrologists in America choose December 1st as the beginning and ending of the water year; others use October 1st, and in England September 1st is the date selected, the object, of course, being to choose a time when the ground-water conditions will be more likely to be the same from year to year.

In the accompanying diagrams, both October 1st and December 1st, have been used, the date for beginning and closing the water year being stated in each case.

Accuracy of Data.—The accuracy of existing data does not justify precision. On some of the territories for which diagrams are presented (Figs. 1 to 19) there is one rainfall station per 1 000 sq. miles of water-shed. In the computation of run-off, a discrepancy of 1 or

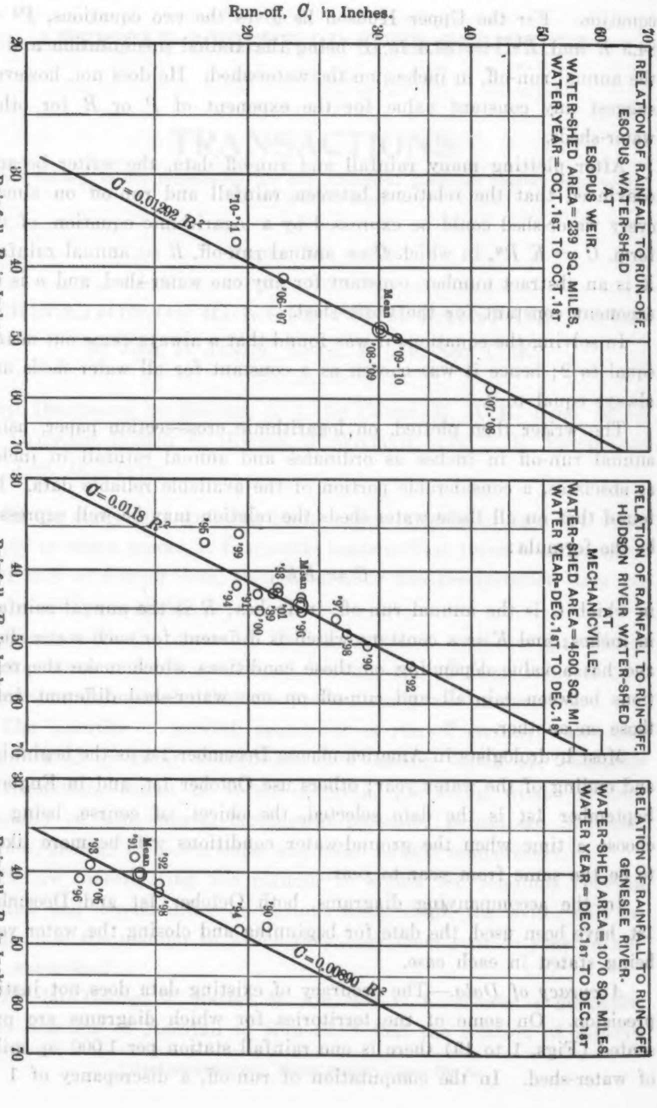
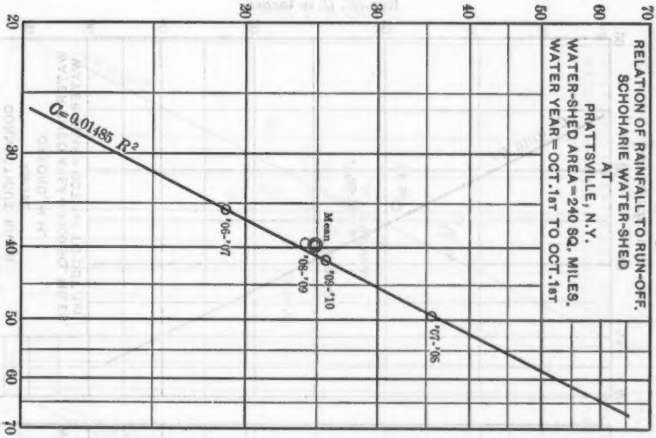


FIG. 1.

FIG. 2.

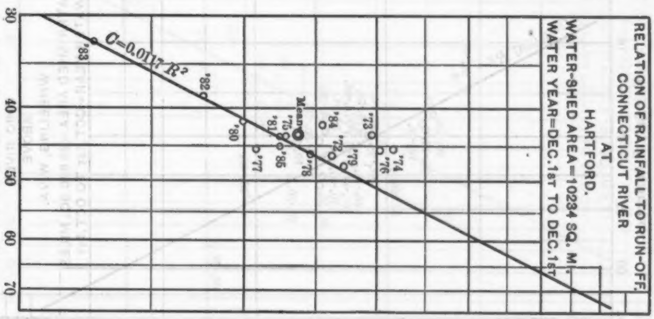
FIG. 3.

Run-off, C, in Inches.



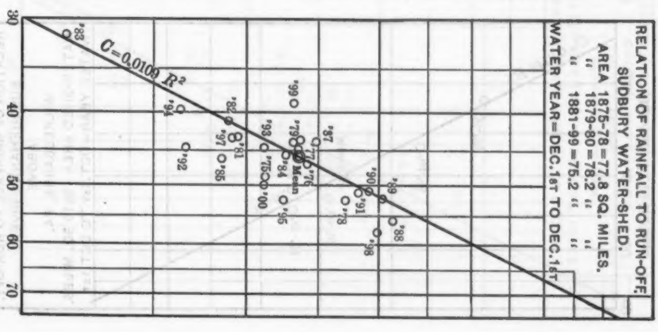
Rainfall, R, in Inches.

FIG. 4.



Rainfall, R, in Inches.

FIG. 5.



Rainfall, R, in Inches.

FIG. 6.

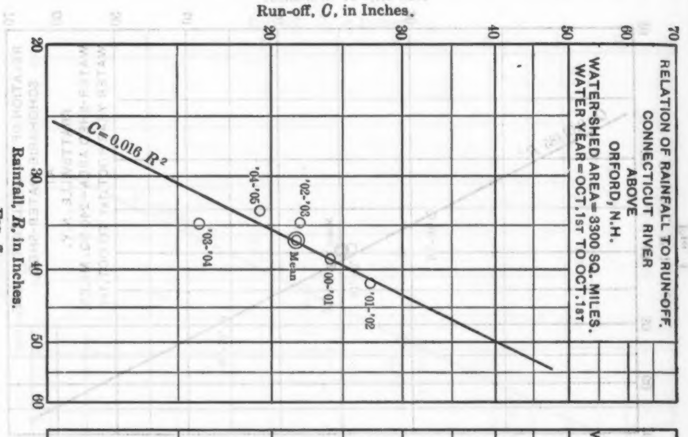


Fig. 7

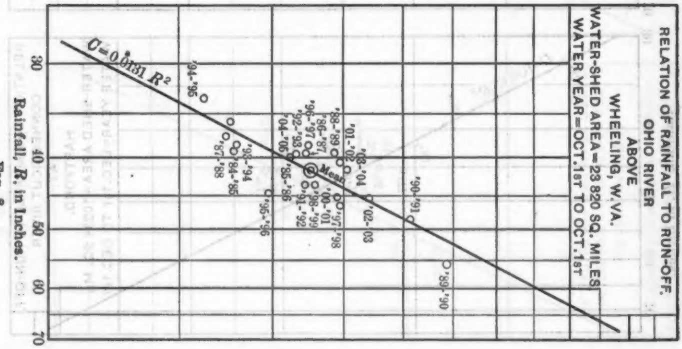


Fig. 8.

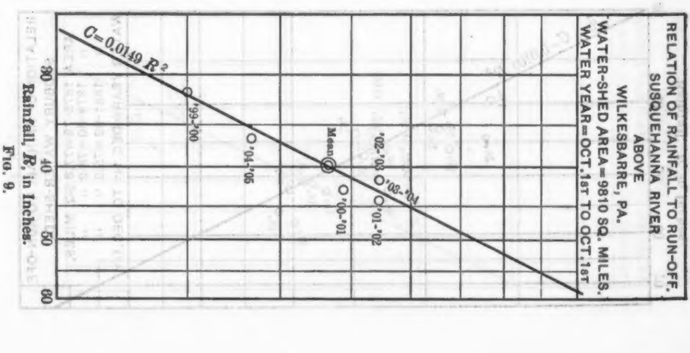
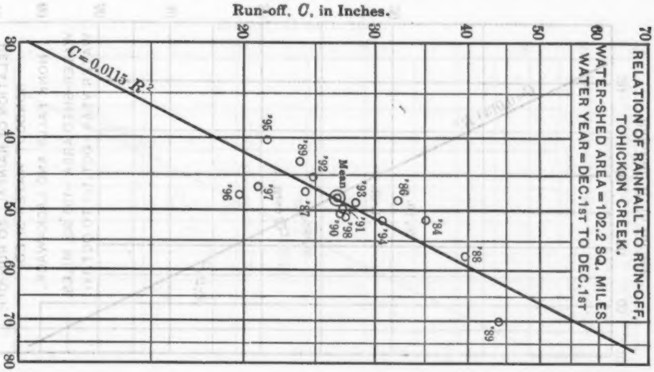
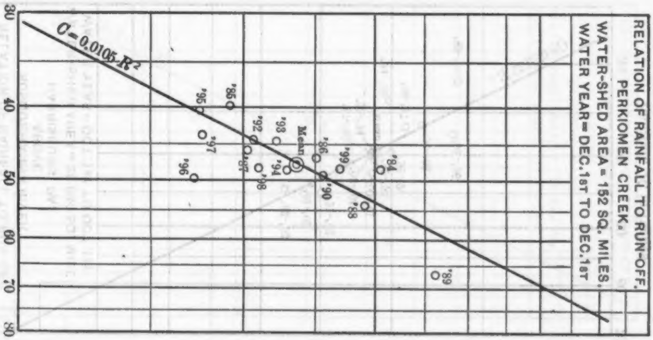


Fig. 9.



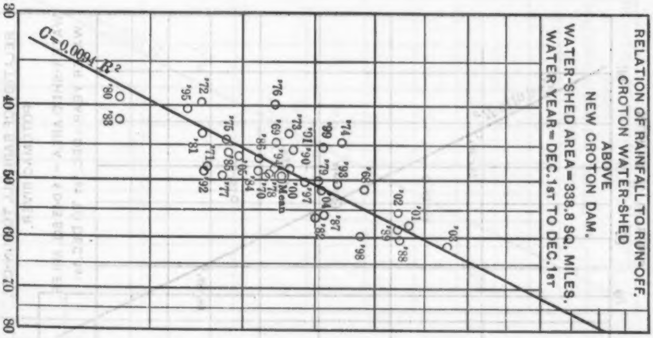
Rainfall, R, in Inches.

FIG. 10.



Rainfall, R, in Inches.

FIG. 11.



Rainfall, R, in Inches.

FIG. 12.

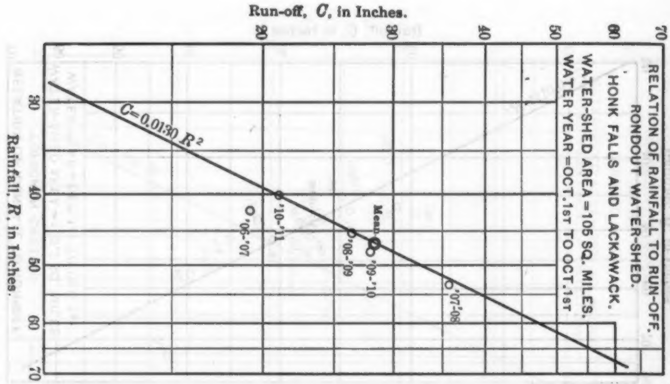


FIG. 13.

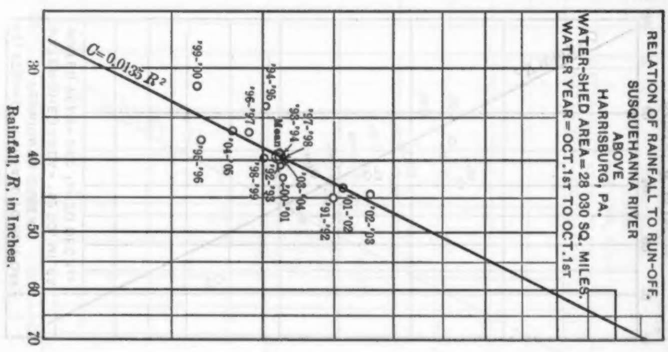


FIG. 14.

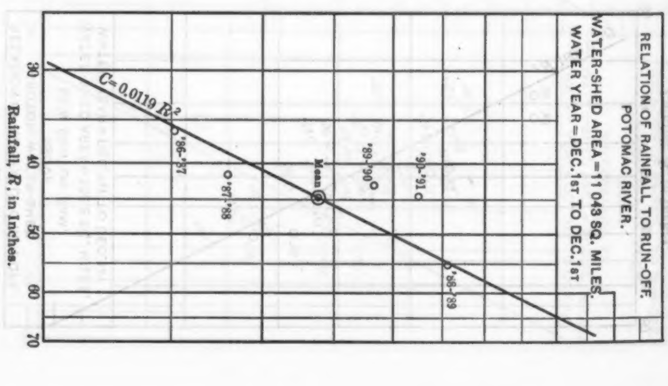


FIG. 15.

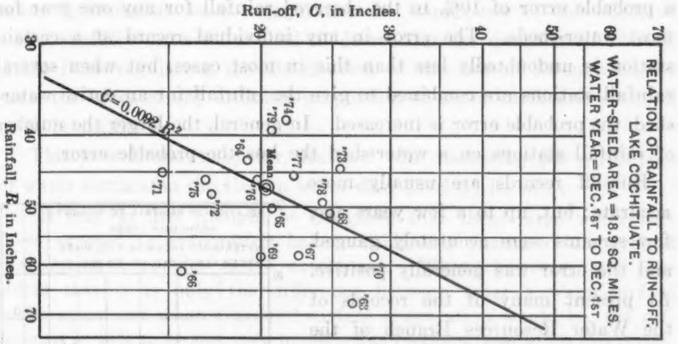


FIG. 16.

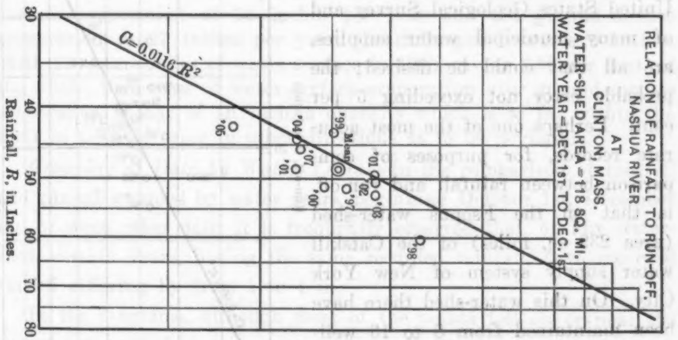


FIG. 17.

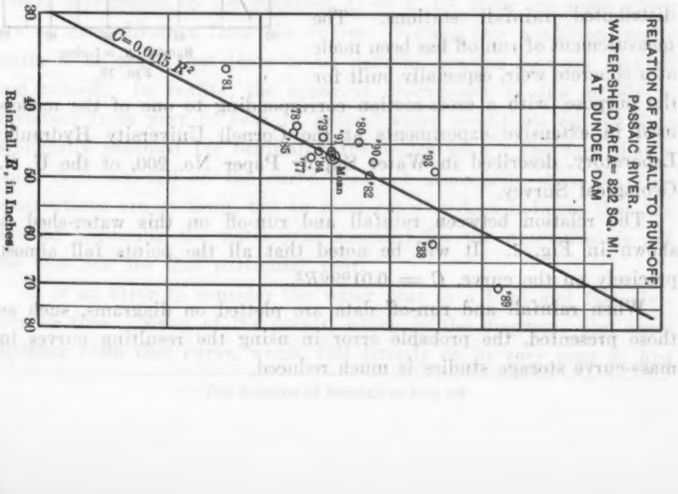


FIG. 18.

2 in. is not worthy of discussion. In most cases one can safely assume a probable error of 10% in the observed rainfall for any one year for most water-sheds. The error in any individual record at a certain station is undoubtedly less than this in most cases, but when several rainfall stations are combined to give the rainfall for an entire water-shed, the probable error is increased. In general, the larger the number of rainfall stations on a water-shed the less the probable error.

Run-off records are usually more accurate; but, up to a few years ago, few streams were accurately gauged, and the error was generally positive. At present many of the records of the Water Resources Branch of the United States Geological Survey and of many municipal water supplies, are all that could be desired; the probable error not exceeding 5 per cent. Perhaps one of the most accurate records, for purposes of comparison between rainfall and run-off, is that on the Esopus water-shed (area 239 sq. miles) of the Catskill water supply system of New York City. On this water-shed there have been maintained from 8 to 13 well-distributed rainfall stations. The measurement of run-off has been made at a concrete weir, especially built for the purpose, with a cross-section corresponding to one of the models used in extensive experiments at the Cornell University Hydraulic Laboratory, described in Water Supply Paper No. 200, of the U. S. Geological Survey.

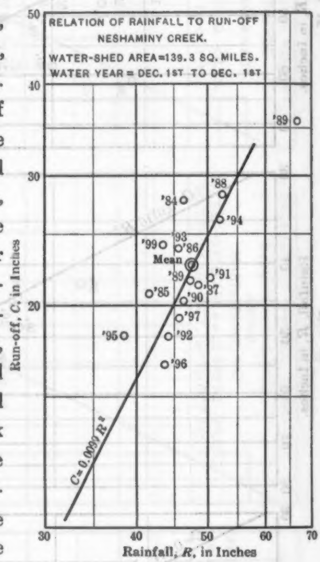


FIG. 19.

The relation between rainfall and run-off on this water-shed is shown in Fig. 1. It will be noted that all the points fall almost precisely on the curve, $C = 0.01292R^2$.

When rainfall and run-off data are plotted on diagrams, such as those presented, the probable error in using the resulting curves in mass-curve storage studies is much reduced.

Effect of Proportion of Water Surface.—The writer believes that many hydrologists have exaggerated the importance of the effect of water surfaces on a water-shed in decreasing the quantity of run-off. Aside from regulating the distribution of the run-off throughout the year, the effect of any ordinary proportion of water surface is so small as to be negligible.

This is well shown by Mr. Rafter.* In discussing the percentage of water surface on the Croton water-shed (3.56%) he says:

"It may at first thought be imagined that these large water surfaces exposed to evaporation have considerably increased the ground evaporation over the entire catchment. When, however, one considers that it is only the difference between what a water-surface evaporation and what a ground-surface evaporation would be, the difference is seen to be not very much. For instance, assuming the water-surface evaporation at 36 inches per year and the ground-surface evaporation at 27 inches per year, the difference becomes 9 inches. With 12 square miles of water surface in 1900, giving 3.56 per cent. of the whole, the excess of water-surface evaporation over ground-surface evaporation is 0.32 of an inch, a quantity which is so far within the limit of possible error in other directions as to be negligible."

Grouping of Data by Water Years.—In the comparison of rainfall and run-off grouped by water years, beginning October 1st, December 1st, or some other date, it is frequently observed that, on any water-shed, certain years, having the same recorded rainfall, have recorded run-off differing by from 1 to 4 in.

In the diagrams, although most of the points fall on or near the curves represented by the equation, $C = K R^2$, there are some which are at some distance from the curves. Noting this condition, many claim it as proof that there is no definite relation between rainfall and run-off. In reality, the apparent discrepancy is due merely to variation from year to year in ground-water conditions on the date arbitrarily assumed for beginning the water year. The fact that a point does not fall on the curve does not necessarily show that the observations are at fault, but in most cases it does indicate that the date of beginning the water year assumed for the water-shed is not the true one for that particular year.

It is an error to consider the water year as a hard and fast division of time. Many of the points, which in the diagrams plot some distance from that curve, would fall directly on or very near it, had

* "The Relation of Rainfall to Run-off."

the first of the preceding or succeeding month been used for beginning the water year.

The true water year does not begin or end at any particular date, but should be regulated so that ground-water conditions are nearly constant on the dates selected for the beginning of such years. It is believed that, if this could be done, all points for which the data are accurate would fall on or near the curve represented by the equation, $C = K R^2$.

Owing, however, to the almost utter lack of data on ground-water levels, it is impracticable to adopt this method of division. Especially in the study of large storage propositions, the apparent discrepancies will balance each other, and will not affect the conclusions.

Manner in Which the Relations Between Rainfall and Run-off Vary from One Water-shed to Another.—It has frequently been observed that, other things being equal, a steep water-shed will have a greater run-off for the same rainfall than a flat one. The water, staying on the water-shed a shorter length of time, has less chance to evaporate. Mr. Vermeule has shown successfully the great influence which the mean annual temperature of a water-shed has on the relation of rainfall to run-off.

These two elements, slope and mean annual temperature, the writer believes, are, in general, the chief factors determining the manner in which the relation between rainfall and run-off vary from one water-shed to another. Hence he will use them in determining the value of K.

The slope of a water-shed may be defined as the difference in elevation between the highest and lowest points divided by the square root of the area.

The proper value for the mean annual temperature of a water-shed can generally be determined by a study of the data published by the United States Weather Bureau in the "Summary of Climatological Data for the United States, by Sections." In using these data, it sometimes happens, especially on mountainous water-sheds, that all the observations are at stations in the valleys. On such a water-shed the mean annual temperature frequently varies directly with the elevation. In such a case, a practical method of determining the mean annual temperature would be as follows:

(1) Take the average temperature at the stations;
(2) The average elevation for the same stations;

(3) The average elevation of the water-shed = the elevation of the highest point plus the elevation of the lowest, divided by 2;

(4) Take the difference between the average elevation of the stations and the average elevation of the water-shed;

(5) Multiply this by the mean difference in temperature per foot of increase in elevation;

(6) Finally, subtracting this from the average temperature at the stations will give the mean annual temperature of the water-shed.

TABLE 1.—VALUES OF K, T, S, AND KT, FOR VARIOUS WATER-SHEDS.

Water-shed.	K in the formula, $C = KR^2$.	T. Mean annual temperature.	S. Slope of water-shed.	$KT = K \times T$.
Lake Cochituate	0.0092	48.5	0.0116	0.446
Sudbury	0.0109	46.0	0.0119	0.501
Hudson	0.0118	41.9	0.0149	0.494
Rondout	0 0130	47.0	0.0664	0.611
Croton	0.0094	49.0	0.0110	0.460
Esopus	0.01292	44.5	0.0468	0.575
Passaic	0.0115	48.4	0.00977	0.555
Genesee	0.0089	45.5	0.00554	0.405
Neshaminy	0.0099	50.6	0.0101	0.500
Perkiomen	0.0105	48.6	0.0148	0.511
Tohickon	0.0115	49.6	0.0166	0.571
Nashua	0.0116	45.0	0.0278	0.522
Connecticut	0.0117	42.0	0.0117	0.490
Schoharie	0.01485	42.3	0.0367	0.630

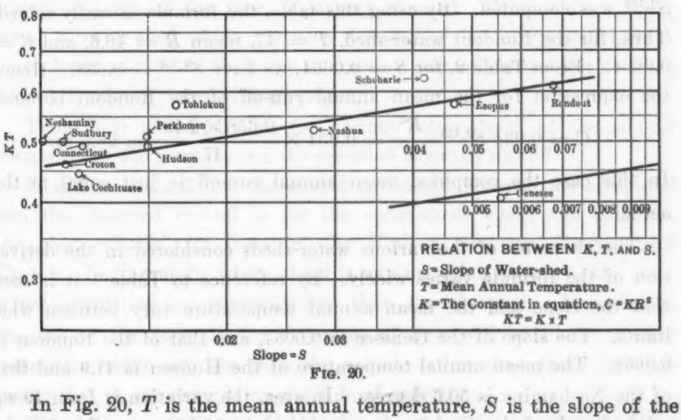


FIG. 20.

In Fig. 20, T is the mean annual temperature, S is the slope of the water-shed, determined as previously described, and K is the constant in the equation, $C = K R^2$.

It was found that for several water-sheds having about the same value of S, K varied very nearly as the first power of T. In Fig. 20 the values of KT (the product of K and T) have been plotted as ordinates and the values of S as abscissas.

Table 1 gives the values of K, T, S, and KT, for various water-sheds.

Picking values off the logarithmic curve in Fig. 20, and solving the equation,

$$K\,T = P\,S^x,$$

P being the unknown coefficient and x the unknown exponent, we have, for the equation of the curve, $K\,T = 0.934\,S^{0.155}$

$$\text{but } C = K\,R^2,$$

$$\text{whence } C = 0.934\,S^{0.155}\,\frac{R^2}{T},$$

which is the general formula for the relation of run-off to rainfall.

C = Annual run-off, in inches, on the water-shed;

R = Annual rainfall, in inches, on the water-shed;

S = Slope of the water-shed, equals the elevation of the highest point minus the elevation of the lowest point divided by the square root of the area;

T = Mean annual temperature of the water-shed, in degrees, Fahrenheit.

For convenience in using the formula, Table 2, giving values of $S^{0.155}$ was computed. By using this table, the formula is easily solved. Thus, for the Rondout water-shed, $T = 47$, mean $R = 46.6$, and $S = 0.0664$. From Table 2, for $S = 0.0664$, we have $S^{0.155} = 0.658$. Hence the expression for the mean annual run-off of the Rondout becomes

$$C = 0.934\,S^{0.155}\,\frac{R^2}{T} = 0.934 \times \frac{0.658 \times 2\,180}{47} = 28.5 \text{ in.}$$

In this case the computed mean annual run-off is just equal to the actual.

The character of the various water-sheds considered in the derivation of the formula varies widely. By reference to Table 1 it is seen that the slope and the mean annual temperature vary between wide limits. The slope of the Genesee is 0.0054, and that of the Rondout is 0.0664. The mean annual temperature of the Hudson is 41.9 and that of the Neshaminy is 50.6 degrees. In area, the variation is from 19 sq. miles for the Lake Cochituate water-shed to 10 234 sq. miles for the Connecticut at Hartford. In the matter of forestation, the variation

TABLE 2.—VALUES OF $S^{0.155}$ FOR VARIOUS VALUES OF S. TO BE USED IN SOLVING THE FORMULA, $C = 0.934\ S^{0.155}\ \frac{R^2}{T}$.

S.	$S^{0.155}$.	S.	$S^{0.155}$.	S.	$S^{0.155}$.	S.	$S^{0.155}$.	S.	$S^{0.155}$.
0.002	0.382	0.0070	0.464	0.0093	0.484	0.026	0.567	0.049	0.628
0.003	0.406	0.0071	0.465	0.0094	0.484	0.027	0.570	0.050	0.630
0.004	0.425	0.0072	0.466	0.0095	0.485	0.028	0.574	0.051	0.632
0.005	0.440	0.0073	0.467	0.0096	0.486	0.029	0.578	0.052	0.634
0.0051	0.442	0.0074	0.468	0.0097	0.487	0.030	0.582	0.053	0.636
0.0052	0.443	0.0075	0.469	0.0098	0.488	0.031	0.584	0.054	0.638
0.0053	0.444	0.0076	0.470	0.0099	0.489	0.032	0.587	0.055	0.640
0.0054	0.445	0.0077	0.471	0.010	0.490	0.033	0.590	0.056	0.641
0.0055	0.446	0.0078	0.472	0.011	0.496	0.034	0.592	0.057	0.642
0.0056	0.447	0.0079	0.473	0.012	0.502	0.035	0.595	0.058	0.644
0.0057	0.448	0.0080	0.474	0.013	0.509	0.036	0.597	0.059	0.646
0.0058	0.449	0.0081	0.475	0.014	0.514	0.037	0.600	0.060	0.648
0.0059	0.450	0.0082	0.476	0.015	0.520	0.038	0.602	0.061	0.650
0.0060	0.451	0.0083	0.477	0.016	0.526	0.039	0.604	0.062	0.652
0.0061	0.453	0.0084	0.477	0.017	0.531	0.040	0.606	0.063	0.653
0.0062	0.454	0.0085	0.478	0.018	0.536	0.041	0.609	0.064	0.654
0.0063	0.456	0.0086	0.479	0.019	0.541	0.042	0.612	0.065	0.656
0.0064	0.458	0.0087	0.479	0.020	0.545	0.043	0.614	0.066	0.658
0.0065	0.459	0.0088	0.480	0.021	0.549	0.044	0.616	0.067	0.659
0.0066	0.460	0.0089	0.481	0.022	0.552	0.045	0.619	0.068	0.660
0.0067	0.461	0.0090	0.482	0.023	0.556	0.046	0.621	0.069	0.661
0.0068	0.462	0.0091	0.482	0.024	0.560	0.047	0.624	0.070	0.662
0.0069	0.463	0.0092	0.483	0.025	0.563	0.048	0.626		

is also large, from the water-shed of the Genesee, with its gently rolling farm lands and few woods, to the heavily forested head-waters of the Rondout and Upper Hudson. The effect of forests on the quantity of rainfall and run-off is believed to be slight, but, on the distribution of run-off throughout the year, it is very marked. Other things being equal, a water-shed which has been denuded of its forests will have a much lower minimum discharge in summer and will be subject to more violent floods in times of high water.

Table 3 is a comparison, for various water-sheds, of the run-off computed by the formula and the recorded observed run-off.

It will be noticed that the closest agreement between the computed and the observed run-off is for the water-sheds where the data are the most reliable, for instance, the Hudson, Genesee, Esopus, and Croton. Furthermore, where there is any material variation, the computed quantities are generally less than the observed. Accordingly, the use of the formula for estimates of flow will be likely to give quantities which are less than the actual, rather than those that are more; that is, the error is on the side of safety.

The formula is also applicable for the computation of run-off for individual years. For illustration, take two water-sheds, the Hudson

and the Esopus. In the case of the Hudson, $S = 0.0149$ and $T = 41.9$, and the formula becomes, $C = 0.934 \dfrac{0.520\,R^2}{41.9}$,

$$\text{or } C = 0.0116\,R^2.$$

In the case of the Esopus, $S = 0.0468$ and $T = 44.5$, and the formula becomes $C = 0.934 \dfrac{0.623\,R^2}{44.5} = 0.0130\,R^2$.

TABLE 3.—COMPARISON BETWEEN OBSERVED RUN-OFF AND RUN-OFF COMPUTED BY THE FORMULA, $C = 0.934\,S^{0.155}\,\dfrac{R^2}{T}$.

Water-shed.	Mean annual rainfall observed, in inches.	Mean annual run-off observed, in inches.	Mean annual run-off computed, in inches.	Difference, in inches.
Rondout	46.7	28.5	28.5	0
Sudbury	45.7	23.6	21.3	2.3
Connecticut	43.5	23.8	21.1	2.7
Lake Cochituate	47.3	20.4	21.5	1.1
Esopus	48.3	30.1	30.4	0.3
Nashua	48.7	25.4	28.2	2.8
Tohickon	48.4	26.7	23.5	3.2
Croton	49.3	22.6	23.0	0.4
Perkiomen	47.6	23.6	22.6	1.0
Passaic	46.8	25.3	20.7	4.6
Neshaminy	47.8	22.8	20.7	2.1
Genesee	40.4	14.3	14.9	0.6
Muskingum	42.4	15.1	13.3	1.8
Hudson	44.5	23.5	22.9	0.6

TABLE 4.—COMPARISON BETWEEN OBSERVED AND COMPUTED RUN-OFF FOR VARIOUS YEARS, FOR THE UPPER HUDSON.

Year ending December 1st.	Rainfall observed, in inches.	Run-off observed, in inches.	Run-off computed by formula, in inches.	Difference, in inches.
1888	43.9	23.6	22.3	1.3
1889	43.0	21.7	21.5	0.2
1890	50.4	28.9	29.5	0.6
1891	43.0	20.6	21.5	0.9
1892	53.9	33.1	33.5	0.4
1893	42.2	21.9	20.8	1.1
1894	42.0	19.4	20.4	1.0
1895	36.7	17.5	15.6	1.9
1896	45.2	23.6	23.7	0.1
1897	46.5	26.2	25.0	1.2
1898	48.5	27.1	27.3	0.2
1899	35.8	19.5	14.9	4.6
1900	45.4	20.7	23.8	3.1
1901	42.6	21.9	21.0	0.9

TABLE 5.—COMPARISON BETWEEN OBSERVED AND COMPUTED RUN-OFF FOR VARIOUS YEARS, FOR THE ESOPUS.

Year ending October 1st.	Rainfall observed, in inches.	Run-off observed, in inches.	Run-off computed by formula, in inches.	Difference, in inches.
1907.........	41.3	22.4	22.2	0.2
1908.........	58.0	42.5	43.8	1.3
1909.........	48.5	30.3	30.5	0.2
1910.........	49.6	31.8	32.0	0.2
1911.........	37.0	19.3	17.8	1.5

These two illustrations serve to show the degree of accuracy that may be expected in the use of the formula when the rainfall and temperature data are fairly accurate.

The Supplying of Missing Records.—It sometimes happens that only 2 or 3 years of good run-off records have been kept on a watershed, and that rainfall records are available for a number of years. In such a case, the record may be extended in the following manner.

Plot on logarithmic cross-section paper points for the 2 or 3 years of known run-off, using rainfall as abscissas and run-off as ordinates. Compute the run-off for several other years, using the formula, $C = 0.934\ S^{0.155}\ \frac{R^2}{T}$. Plot the corresponding points. Then draw a straight line parallel to the line, $Y = X^2$, among these points, giving greater weight to the observed data. This line will be represented by the equation, $C = K\ R^2$. Having this curve, the run-off for any year may be read off, using the rainfall as argument.

Application to the Mass-Curve.—Engineers are frequently called on to construct reservoirs of considerable capacity on water-sheds where accurate run-off data are lacking. For the determination of the necessary capacity of a proposed reservoir, the mass-curve is the accepted method. Having given a water-shed without run-off data, but with rainfall records available, a satisfactory mass-curve may be constructed by using the formula, $C = 0.934\ S^{0.155}\ \frac{R^2}{T}$. The values of S and T are generally easy of determination for any particular water-shed.

In using this formula for the construction of a mass-curve, it is applied to the rainfall for each month consecutively, and the resulting monthly run-off is used in constructing the mass-curve in the usual manner. Of course, using the formula in this way, the resulting com-

puted run-off for individual months will often be very different from the actual; but, in the study of large water-power or water-supply projects, where water must be stored for several months, this will not affect the conclusions as to the necessary size of the reservoir for a given draft.

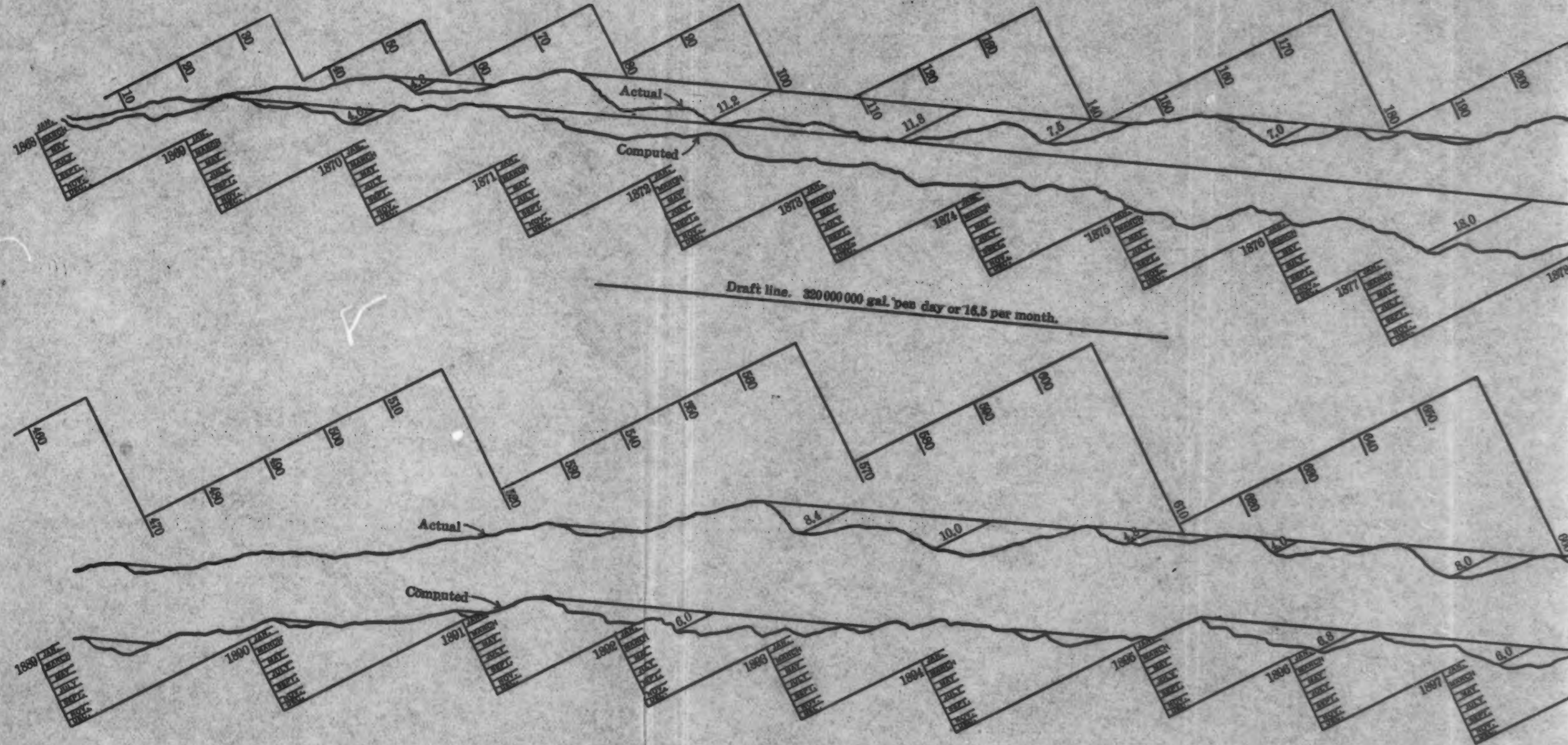
The most expeditious method of applying the formula for this purpose is as follows:

Compute several values of C for given monthly values of R. Plot these computed values of C, with the corresponding values of R, on logarithmic cross-section paper. The points will lie in a straight line. This logarithmic curve may then be used for picking off values of C for given values of R.

On Plate V are shown two mass-curves for the Croton water shed. The upper curve was obtained from the recorded run-off data, and the points on the lower curve were computed by the formula, $C = 0.934\ S^{0.155} \frac{R^2}{T}$. It will be noticed that the cumulative totals of the observed run-off for 37 years differ from the computed by about 3%, the computed being less than the observed. A draft on the water-shed of 1.65 in. per month, or 320 000 000 gal. per day, was assumed and applied to the curves. The greatest depletion shown on the curve of observed run-off was 14.2 in., and the mass-curve of computed run-off showed a depletion of 18 in., a difference of 21 per cent. This difference, however, is on the safe side. It is customary to build reservoirs with a capacity of from 20 to 30% in excess of the depletion shown by the mass-curve. Had there been no run-off data in existence on the Croton water-shed, a reservoir of sufficient capacity could have been decided on from a study of the computed mass-curve.

In a similar manner, two mass-curves were plotted for the Nashua water-shed, one from the observed run-off data and the other from computed run-off data obtained by using the formula. A draft of 1.852 in. per month, or 125 000 000 gal. per day, was assumed. The greatest depletion on the curve of observed run-off was 10 in., and on the curve of computed run-off, 12 in., an error which is again on the safe side.

The mass-curves of observed run-off and of computed run-off were plotted for the Esopus water-shed. A draft of 1.825 in., or 250 000 000 gal. per day, was assumed. For the curve of observed run-off, the


Actual
Computed
Draft line. 320 000 000 gal. per day or 16.5 per month.
Actual
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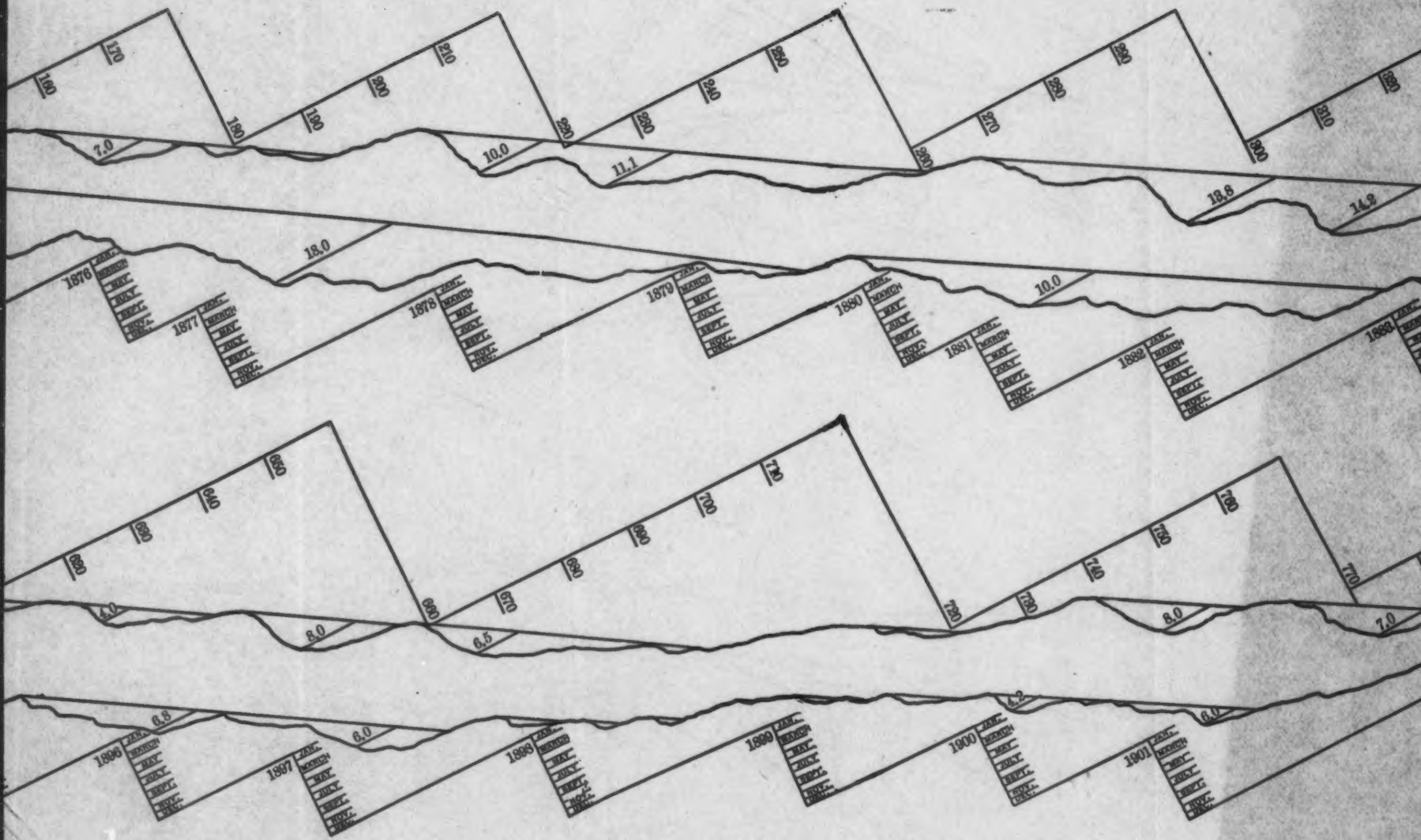

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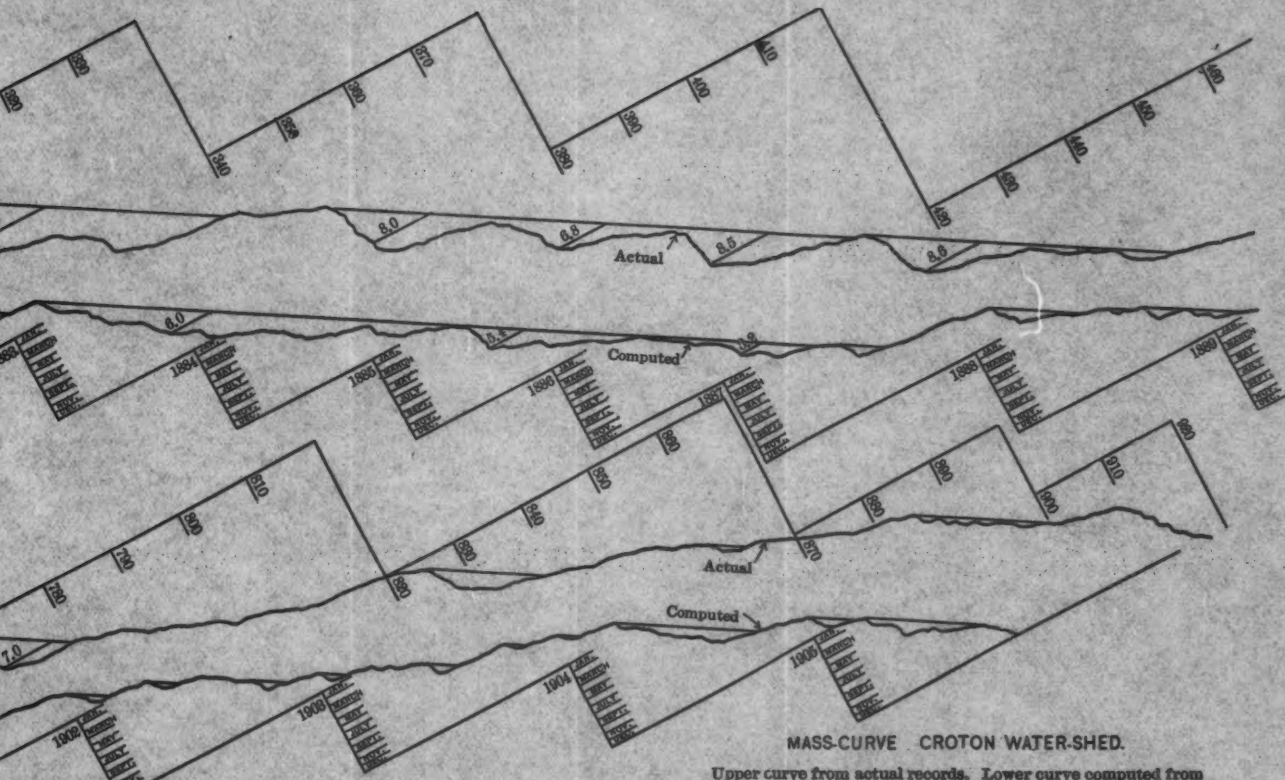
PLATE V.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVII, No. 1288.
JUSTIN ON
DERIVATION OF RUN-OFF FROM RAINFALL DATA.



MASS-CURVE CROTON WATER-SHED.

Upper curve from actual records. Lower curve computed from rainfall, using the formula, $C=0.934\,S^{0.155}\frac{R^2}{T}$, in which C=run-off S=slope, T=mean annual temperature, R=rainfall.
S for Croton=0.0110. T for Croton=49

greatest depletion was 8.2 in., and, for the curve of computed run-off, the greatest depletion was 7.5 in. The difference, 0.7 in., though not on the safe side, is so small that it could not affect materially the size of the reservoir decided on.

Conclusions.—The writer is not of the opinion that the gauging of streams and the accumulation of run-off data should be abandoned, and the method herein described established in their place. Accurate run-off data are scarce, and engineers need far more. These methods and formulas are applicable to water-sheds where run-off data are meager or lacking, and it is believed that they will give more reliable results than those now generally in use.

The formula, $C = 0.934\, S^{0.155} \frac{R^2}{T}$, is, the writer believes, applicable to the Eastern United States, and, in general, should give results within 10% of the true run-off. In applying the formula to other water-sheds, the writer would advise caution. Although the formula is believed to be general, it is possible that, if more data were at hand on the relation of run-off to rainfall, the value of the constant (here 0.934) and of the exponent of S (0.155) might vary somewhat in other sections of the country, where there is a marked difference in climatic conditions.

Of course, if this formula is applied to the rainfall for some particular month it will not give the true run-off. It has been shown, however, that it may be used for obtaining monthly run-off, and that the mass-curve when plotted gives depletions which do not differ materially from those obtained when the observed monthly run-off is used in constructing it.

At first sight, the application of the formula may appear to be complicated, but, by using Table 2, which gives the values of $S^{0.155}$, it is simple; and if logarithmic plotting is utilized, as suggested, it becomes merely a matter of reading off the curve.

DISCUSSION

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The formula presented by the author is very ingenious and seems to fit the cases considered with surprising closeness. Where the basin is for water supply and power purposes, however, the writer would particularly caution the young engineer against making any practical use of the long average run-off, which is that furnished by the formula.

The water supply storage reservoirs at and near San Francisco, Cal., have been in constant use for 43 years, hence the practical results obtained are very interesting. The long average rainfall is 40 in., and the long average run-off is 8 in., or 20 per cent. The important fact remains, however, that at intervals of 10 or 12 years there are 3 years in succession in which the rainfall is only 18 or 20 in., and is distributed so uniformly that the thirsty ground takes it all, and, as a result, there is absolutely no run-off for 600 days or more. The reservoirs, therefore, have to be large enough to hold a 3 years' supply without any possible hope of replenishment.

The same important feature is noticed on the Sudbury water-shed,* where the record for 37 years shows a long average yield of 1 000 000 gal. per day per sq. mile of water-shed; but, during the three successive years, 1909, 1910, and 1911, the average yield was only 550 000 gal. per day per sq. mile. This is practically the limit of usefulness for this water-shed. The record shows that in 1888 the average yield was 1 700 000 gal. per day per sq. mile of water-shed, but as this only happens once in about 25 years, this isolated fact has no practical value for water-supply purposes.

Mr. Begg. R. B. H. Begg, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Justin's paper is an interesting addition to an important subject, but the writer would like to call attention to the danger of attempting to apply any general formula to so complicated a matter as the relation of rainfall to run-off. The writer also wishes to present a hitherto unpublished run-off curve (Fig. 21), which may be of interest. This curve shows the relation of run-off to rainfall for the principal water-sheds of Kansas. The run-off figures were taken from the stream guagings of the U. S. Geological Survey, and the rainfall from the reports of the U. S. Weather Bureau. The points plotted, with very few exceptions, fell very close to the final curve. The principal point of interest in this curve is the great difference between it and the curves for the Northeastern States, which are the ones usually seen.

The conditions governing the run-off in Kansas are quite different from those in the East; of these the two most important are the

* Twelfth Annual Report, Metropolitan Water and Sewerage Board of Massachusetts, 1912, p. 58.

slight slope of the drainage areas and the fact that the greatest rainfall occurs in summer, when vegetation is most abundant, and, consequently, evaporation from the ground is greatest. It will be noticed that, for a rainfall of 40 in. on the water-sheds mentioned by Mr. Mr. Begg.

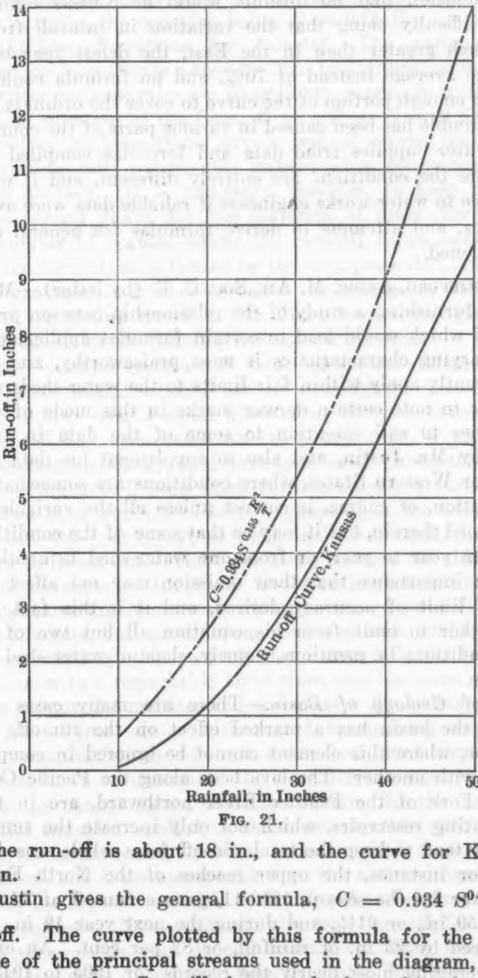


FIG. 21.

Justin, the run-off is about 18 in., and the curve for Kansas gives only 6.1 in.

Mr. Justin gives the general formula, $C = 0.934\ S^{0.155}\ \frac{R^2}{T}$, for the run-off. The curve plotted by this formula for the Republican River, one of the principal streams used in the diagram, Fig. 21, is on the same scale. It will be noted that this formula gives

Mr. Begg. results about 40% greater than those obtained from the curve plotted from actual measurements for rainfalls of 30 in. or more, and for smaller rainfalls it gives even greater discrepancies. The writer attempted unsuccessfully to derive a formula that would fit this curve, and came to the conclusion that no formula would fit Kansas conditions, the principal difficulty being that the variation in rainfall from year to year is much greater than in the East, the driest year being about 50% of the average instead of 70%, and no formula could be found to fit a long enough portion of the curve to cover the ordinary variations.

Much trouble has been caused in various parts of the country by calculating water supplies from data and formulas compiled for use in places where the conditions are entirely different, and it would be of great service to water-works engineers if reliable data were available for all localities, and attempts to derive formulas for general application were abandoned.

Mr. Clifford. R. G. CLIFFORD, ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Justin's attempt at furnishing a study of the relationship between precipitation and run-off which would lead to certain formulas applicable to watersheds of varying characteristics is most praiseworthy, and his deductions apparently apply within fair limits to the water-sheds considered.

In order to note certain danger marks in this mode of attack, the writer wishes to call attention to some of the data in the records furnished by Mr. Justin, and also to supplement his data with some from the far Western States, where conditions are somewhat different.

No equation, of course, is correct unless all the variables involved are introduced therein, but it may be that some of the conditions which change from year to year, or from one water-shed to another, are of such minor importance that their omission may not affect the result beyond the limit of accuracy desired, and it is this fact which has led the author to omit from his equation all but two of the eight variable conditions he mentions, namely, slope of water-shed and mean temperature.

Effect of Geology of Basin.—There are many cases where the geology of the basin has a marked effect on the run-off, as on the Pacific slope, where this element cannot be ignored in comparing one water-shed with another. The lava beds along the Pacific Coast, from the North Fork of the Feather River northward, are in themselves huge regulating reservoirs, which not only increase the summer flow, but actually tend to keep the total run-off for a whole season near the normal. For instance, the upper reaches of the North Fork of the Feather River for the season, 1910-11, gave a run-off of 36 in. with a rainfall of 59 in., or 61%, and during the next year 19 in. of run-off were produced by 23 in. of rainfall, or 82 per cent. An exponential equation averaging most nearly the records for 1905 to 1913 for this

water-shed would be $R = 2.2\ P^{0.7}$ (R being the run-off and P the precipitation). Mr. Clifford.

Farther south, along the Sierra water-sheds, where granite peaks have replaced the lava beds, the exponential varies up to $P^{1.5}$. For the South Fork of the Yuba River at the Spaulding Dam, the average exponent is unity, the equation being $R = 0.74\ P$.

Straight-Line Equations for Run-Off.—The various water-shed data drawn on logarithmic paper by Mr. Justin, having been replotted by the writer on natural-scale co-ordinate paper, it is found that a straight-line equation of the form, $R = a\ P + K$, will fit all of them just as well as the logarithmic curves, but, of course, the introduction of two variables only complicates matters when comparing one water-shed with another. It is necessary, however, to keep the exponent below the second power for the majority of cases, for there are a limited number of regions where the rainfall varies only 20% above or below the normal, as indicated by the data presented. On the Pacific slope the variation is from 40% below the average to 50% above, and the use of the second power in Mr. Justin's equation gives values of run-off in excess of the rainfall for the wetter years.

The very marked variation in total run-off for different years having the same rainfall should not be overlooked. The records from Lake Cochituate, with a small water-shed, show an extreme case, and if the run-off had been based on the years, 1865 to 1872, inclusive, a very different constant would have been used. Even the large water-shed above the new Croton Dam furnishes such a variety of points that no two men would be likely to choose just the same line as an average, and the years, 1872 to 1880, inclusive, would form the basis for a very different one. There are many reasons for this variation in run-off for the same precipitation. As the author points out, the ground-water conditions and inaccuracy in true average rainfall are probably important factors, but, in addition, there are such modifying elements as dry winds and relative packing of the snowfall; the former cutting down the snow in a remarkably short time, and the more solid the snow pack, the slower the melting and the greater the percolation.

The Mass-Curve.—It is with regard to applying any formula to monthly precipitation on different water-sheds, and thereby constructing a mass-curve to determine the necessary storage, that the writer feels that he cannot agree with Mr. Justin. In the first place, the exponential equation applied to the small monthly precipitations will not give the same yearly total as when applied to the total yearly precipitation, and it seems probable that the author intended merely to proportion the monthly rainfall to that for the whole year as computed from his formula. Now, although the total run-off for a season for different water-sheds may depend mainly on mean temperature and slope, yet the ratio of summer flow to winter flow may be absolutely

Mr. Clifford.

different. For instance, there are two branches of the same river, each having a water-shed of approximately 500 sq. miles and the same mean temperature, and yet the summer and winter flows of one have been, respectively, 560 and 10 000 cu. ft. per sec., though the second branch, in the same year, discharged 250 cu. ft. per sec. in summer and reached a maximum of 80 000 cu. ft. per sec. for the same winter flood as the other.

This is undoubtedly a peculiar case, and is principally due to the geological formation, but, to some extent, it is also due to the fact that there is a greater proportion of one water-shed at the higher elevation receiving the precipitation in the form of snow.

A study of the actual and computed mass-diagrams of the Croton water-shed is hardly a convincing argument in favor of this mode of determining the storage needed, unless a very long time average is used and it is feasible to build a reservoir large enough to effect regulations over long dry periods.

Considering the draft line used for the Croton water-shed, if in 1880 a dam had been constructed based on the rainfall formula, a depletion of 18 in. would have to be provided for; and, if 20 to 30% in excess were allowed, there would have been a safety factor, over actual conditions to date, of 60%, which would have been rather extreme.

In a great many power projects it is either physically impossible or very uneconomical to store much more water than comes from the average low years, and in such cases the essential feature is to know just how much auxiliary power is required for the extreme years and how often they come. The safest procedure, if no river data at all have been kept, is to study the run-off from the nearest available water-shed, having a similar geological drainage basin, on which records can be obtained, using Mr. Justin's corrections for slope and mean temperature. A percentage allowance for extreme conditions can then be based on a study of such long-time results as given on the Croton mass-curve.

To obtain the storage needed for power purposes, the writer has used an empirical formula based on the available data for the South Yuba River, which, for that particular water-shed, checks remarkably well for the 8 years available. This formula has not yet been applied to any other water-shed, and so would be most unsafe to use elsewhere. The basis for the formula is the length of the dry season and the precipitation. It pre-supposes that run-off records have been kept carefully for at least one year, the storage needed for any other year being in this particular case proportional to the length of dry season corrected inversely by one-third of the difference in percentage, from the average precipitation of the season immediately preceding. By using the equation, $R = 0.74\ P$ (applicable to this river), the high

points on a mass-curve can be located, and the low points can be plotted by means of the storage computed for each year, thus permitting of storage over long periods being found. The length of dry season is the most indefinite part of the proceeding, being the time of low stream flow, and is found approximately by records on other streams with similar geological formation, or, if a few years of monthly run-off and precipitation are available, by plotting these together and observing how the low-water flow follows the last heavy rains and picks up after the fall or winter storms. Mr. Clifford.

ROBERT E. HORTON, M. AM. SOC. C. E. (by letter).—Mr. Justin presents the formula: Mr. Horton.

$$C = 0.934\, S^{0.155}\, \frac{R^2}{T}$$

$$S = \frac{E \text{ max.} - E \text{ min.}}{A}$$

S = Average absolute slope of the drainage basin, in feet per foot;
E Maximum = Maximum elevation at any point in the drainage basin;
E Minimum = Minimum elevation, or elevation at outlet;
A = Area of drainage basin, in square miles;
R = Annual precipitation, in inches;
T = Mean annual temperature, in degrees, Fahrenheit;
C = Annual run-off depth, in inches.

He adopts the suggestion of the late George W. Rafter, M. Am. Soc. C. E., for the use of an exponential relation between rainfall and run-off. Mr. Rafter applied similar formulas to individual drainage basins, but as the slope, ground-water, and cultural conditions of each basin were more or less constant, it was not necessary for him to include these factors separately in his formula. The author's formula represents an attempt to derive a general relation between rainfall and run-off on different drainage basins, taking into account differences in mean temperature and slope, and he claims that this formula is applicable to the determination of the mean run-off throughout a series of years for streams in the Eastern United States. Some claim is also made that it is applicable to the determination not only in individual years but in individual months. The writer believes that the formula has a certain degree of value for practical application, but that its use should be confined to streams having relatively large drainage areas and with hydrological conditions similar to those prevailing in the larger streams of the New England and Eastern States.

Mr. Justin's study, though perhaps too largely statistical, contains many valuable suggestions. The writer does not feel that the formula given is entirely rational or that it will give reliable results in the one

Mr. Horton. instance where a practicing engineer commonly requires an estimate of the annual yield of a stream. Very many of the larger streams in the East have been accurately gauged. A large proportion of the public water supplies in the Eastern States are derived from small drainage basins which have not been accurately gauged. In the case of such basins, extensive storage is usually developed, and engineers are frequently called on to determine the minimum annual yield available to fill storage reservoirs. In such cases the mean annual yield for a period of years is of little use. Mr. Justin's formula does not appear to fulfill this requirement, because it certainly is not capable of general application to small drainage areas, say less than 25 sq. miles in extent. The absence of any factor in the formula to take into account differences in drainage basins as regards cultural and evaporation conditions is notable. Errors in calculations of the run-off for individual years, as suggested by the author, can be partly eliminated by starting the water year in each case with the ground-water and surface storage reduced to the same datum.

The author's formula is equivalent to the assumption that the annual evaporation loss is a direct and constant function of the rainfall or precipitation for areas having the same mean temperature. He presents a table of the calculated mean annual run-off from various drainage basins which shows surprisingly good agreement with his formula, and would appear to bear out the above conclusion. This conclusion is probably approximately correct for relatively large areas, each area containing a wide variety of different cultural conditions; the differences also make it evident that the annual evaporation loss may differ by considerable amounts when comparing small areas having the same rainfall, mean temperature, and slope, but differing widely as regards cultural conditions.

As a rule, in the case of streams in the Northeastern and New England States, about three-fourths of the annual run-off occurs during the winter or frozen season, when differences in soil and cultural conditions have but little effect. For example, given two drainage basins, each having 18 in. of run-off during the frozen season; the rainfall, slope, and temperature being the same in each, the evaporation loss during the vegetation season being 100% greater in one basin than in the other, giving, during the vegetation season, say, 6 in. annual run-off in one basin and 9 in. in the other, the difference in the total annual run-off for the two streams would be only $12\frac{1}{2}$ per cent. This example illustrates and explains the fact that, in the application of a formula of this kind, differences in soil and cultural conditions are much less important than might at first appear to be the case; it also illustrates the danger of wide errors appearing when an attempt is made to apply such a formula to individual months or even years.

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The author's formula for computing the mean slope of a drainage basin is very simple, but it does not appear to be sufficiently accurate. Furthermore, it appears probable that the variation in run-off, which, in his formula, the author attributes to slope, is not due to that alone, and, in fact, there may be other conditions not directly taken into account, having greater weight as regards their effect on annual run-off, which possibly may have masked the true effect of slope almost entirely in the case of the streams used by the author in deriving his slope formula. It is quite generally true that the flatter portions of a drainage basin are built-up, cleared, drained, and cultivated, whereas the steep slopes are more likely to be forest covered. In the author's method of derivation of the relation of slope to run-off, the effect of prevailing differences in cultural conditions between flat and steep areas would appear in the result as part of the effect of slope difference.

As regards the effect of forest conditions, the author appears to be of the opinion that it makes little difference in the annual run-off whether or not a drainage basin is forest covered. In the writer's opinion, Mr. Justin has omitted to mention the most important effect of forests in relation to run-off, as regards his formula. In fact, his conclusion, that deforesting a drainage basin would tend to reduce the low-water flow and at the same time greatly increase the floods, disproves his other contention. Assume, for example, the total dry-season flow of a stream to be 6 in.; let this be reduced to 3 in. by deforestation; also assume the flood period run-off with forest to be 16 in.; and let this be increased as a result of deforestation to 24 in. The net result will be an increase of 5 in. in the total annual run-off of the stream, or nearly 23%, due to deforestation. In the writer's opinion, this represents fairly a state of affairs which frequently occurs. However, the writer does not agree with Mr. Justin that the general or universal effect of deforestation is a decrease in the low-water flow of a stream; that may or may not be the result, depending largely on the permeability of the soil. For the Northern and Eastern States, the writer believes that the apparent increase in low-water flow of streams in the presence of forests is largely due to retardation of the melting of snow in the spring. This shortens the low-water season. On the other hand, the elaborate European forest experiments seem to prove conclusively that, taking into account intercepted rainfall, transpiration, and evaporation from the soil in the forest as compared with evaporation from the soil alone in the open, there is much greater loss of water during the growing season with forest than without. This proposition is confirmed by extensive European experiments showing that the ground-water table is generally lower under a forested area than under a cleared area.

With reference to the author's formula for the slope of a drainage basin, consider an inclined plane similar to Fig. 22. For purposes of

Mr. Horton.

illustration, let the slope of the plane be unity. For the three areas, *a b x o, c d x o*, and *e f x o*, Mr. Justin's formula gives slopes as follows:

Area, *a b x o*,

$$S = \frac{1 - 0}{\sqrt{1}} = 1$$

Area, *c d x o*,

$$S = \frac{2 - 0}{\sqrt{2}} = 1.42$$

Area, *e f x o*,

$$S = \frac{3 - 0}{\sqrt{3}} = 1.734$$

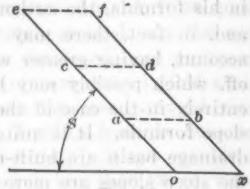

FIG. 22.

Thus it appears that, for an area on which the slope is actually constant, his formula gives a variation in slope of 75%; and although his formula is correct for square inclined planes, it is incorrect for planes which are not square. In other words, the apparent slope, as determined by his formula, is dependent on the shape of the area. For example: Compare square and circular plane areas having unit slope and unit area. For the square area, the author's formula gives $S = 1$, which is correct. For the circular area, it gives $X = 1.13$, an error of 13 per cent. For square areas which are not inclined planes, the author's formula may be seriously in error. For example: Consider the area shown in Fig. 23, which is assumed to represent a drainage basin of 9 sq. miles. According to his formula, the average slope of this basin is 116.67 ft. per mile. The actual average slope is 200 ft. per mile.

It appears that in the case of drainage basins deeply serrated by gullies, and long and narrow in form, the true slope may differ widely from that indicated by his formula. Such areas are very common in the hillside drainage of large valleys, such for example as the Upper Mohawk Valley, or the Finger Lakes, Flint Creek, and Cayuta Creek drainage basins, in the central part of New York State.

The writer suggests the following considerations as a basis for determining more accurately the slope of a drainage basin. The method given can be applied without difficulty or excessive labor in cases where the topographic maps of the U. S. Geological Survey are available. Referring to Fig. 24, consider the drainage basin crossed by contour lines representing equal differences in elevation. Let *a-b* represent the average distance between the contours *a* and *b*; let *D* equal the contour difference of elevation, in feet; then, as the map is a horizontal projection of the area,

$$\frac{D}{a\ b} = \tan. S$$

where *S* is the angle of the slope. The average slope of the strip between the contours, *a* and *b*, is equal to the contour difference, *D*,

divided by the average width of this strip, and a similar relation holds for the entire area. Mr. Horton.

Let W = the average horizontal distance between contours for the entire area;

L = Total length of contours on the area;

A = Area of the drainage basin.

$$\text{Then } W\,L = A$$

$$\tan. S = \frac{D}{W} = \frac{D\,L}{A}.$$

Where topographic maps are available, L may be determined by measuring with an opisometer the length of contours at 20-ft. or 40-ft. intervals, for small or flat areas, and the length of contours of 100-ft. to 500-ft. intervals for large or steep areas.

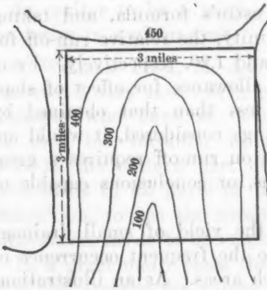

FIG. 23.

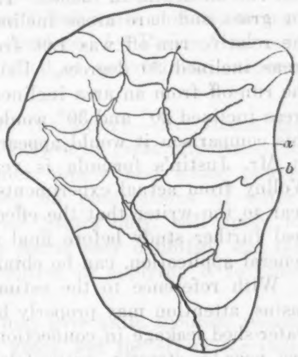

FIG. 24.

In the writer's opinion, the effect of slope on run-off is not measured entirely by the average slope of the topographic surface, but rather by the average slope of the lines which the water travels from the point where the rainfall strikes the ground to the point in the stream to which the area is taken. For an inclined plane, the average effective slope, if so it may be called, measured in this way, would not be the same as the average topographic slope of the surface. The effective slope at any point would be the elevation of the point divided by its distance from the point of measurement of the area. For geometrical figures, the mean distance of all points on an area from a point on the perimeter of the area can be determined from the surface integral of the area.* For example: In the case of a circular inclined plane, the shortest average distance which the water

* Byerly, "Integral Calculus," pp. 201-209.

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can travel to reach a point on the circumference would be 1.273 multiplied by the radius of the circle.

It would appear to the writer that the effect of slope on run-off can be better determined from a comparison of the run-off of small areas having similar topographic and cultural conditions throughout, than from the comparison of such large areas as have been used by Mr. Justin. In confirmation of this opinion, the elaborate and carefully conducted experiments of Wollny may be cited.* Throughout two growing seasons, 1882 and 1883, Wollny determined the run-off from areas of 1 sq. m. of argillaceous sand containing humus. In one series of experiments he used areas inclined 10°, 20°, and 30° to the horizontal and covered with a growth of grass. In another series of experiments he used similar inclined bare areas. The writer has reduced the total run-off for the period covered, April to September, inclusive, to equivalent run-off depth, in inches. Taking the mean run-off for the 2 years for grass and bare areas inclined 10° as a unit, Wollny found that the relative run-off was 1.66 from areas inclined 20°, and 2.40 from areas inclined 30 degrees. Using Mr. Justin's formula, and taking the run-off from an area inclined 10° as unity, the relative run-off for areas inclined 20° and 30° would be 1.12 and 1.20, respectively. From this comparison it would appear that the allowance for effect of slope in Mr. Justin's formula is very much less than that obtained by Wollny from actual experiments. All things considered, it would appear to the writer that the effect of slope on run-off requires a great deal further study before final conclusions, or conclusions capable of general application, can be obtained.

With reference to the estimation of the yield of small drainage basins, attention may properly be called to the frequent occurrence of water-shed leakage in connection with such areas. As an illustration: two near-by streams, accurately gauged by weirs under the writer's direction for several years, showed for the one a minimum run-off of about 250 000 gal. from a topographic drainage basin of 0.4 sq. mile, and for the other a minimum run-off of zero for a period of from 1 to 4 months in nearly every summer, with a drainage basin of 1.18 sq. miles. The topographic run-off from the two streams is probably about the same, but the effect of the inversion of water to one basin by water-shed leakage is sufficient to produce a considerably larger total annual run-off for that basin.

According to Mr. Justin's formula, the annual run-off varies inversely as the mean annual temperature. A method of estimating run-off involving the temperature as a factor was developed by Mr. C. C. Vermeule, a number of years ago. The thing which varies with the temperature is the evaporation loss, but the writer believes that the quantity evaporated is only indicated in a somewhat rough and irregu-

* *Forschungen auf dem Gebiete der Agricultur-Physik*, Prof. E. Wollny, Band XVIII, 1885, pp. 486, 516.

lar manner by the temperature. The evaporation loss is a function of the character and condition of vegetation and of the difference between the absolute vapor pressure in the atmosphere and the vapor pressure corresponding to the temperature of the evaporation surface. A number of years ago, Mr. Rafter pointed out that the evaporation loss is much more nearly constant than the rainfall. The reason for this does not appear to have been given. Apparently, however, the explanation lies in the fact that wet years are usually relatively cold and dry years are usually relatively hot. The evaporation also varies materially according to the distribution of the rainfall. In a hot dry year evaporation may be less than in a colder but wetter year, for the reason that during periods of extreme drought there is very little exposed surface, except in lakes or ponds, from which evaporation can take place. Mr. Horton.

The determination of evaporation losses by proportionate ratios deduced from measured evaporation from a free water surface, as suggested by Penck and more elaborately developed by Parker,* appears to the writer to be a more rational method of ascertaining evaporation losses in a given year.

J. WILLIAM LINK, M. AM. SOC. C. E. (by letter).—Mr. Justin has presented a very interesting paper on a subject of great importance to engineers who have to deal with water supply and water power, but that his formula, which is based on observations of water-sheds in the eastern part of the United States only, will apply to water-sheds in all parts of the country seems very doubtful, and his admonition, to use the formula with caution in applying it to other water-sheds, needs to be emphasized. Mr. Link.

In a great many cases the formula may give quite accurate results as to the average for a long period of years, but the errors for individual years may be quite large, and the young engineer or the engineer unfamiliar with handling rainfall and run-off data, should be extremely careful in its use. As an illustration of this point, reference to the author's diagram for the Sudbury water-shed will show that the actual run-off in several years varies as much as 8 or 10 in. from the results obtained by using the curve.

The author suggests that in the computation of run-off a discrepancy of 1 or 2 in. is not worth discussion. This may be true in some instances, though in considering the yield of a water-shed as related to water-power possibilities, even this discrepancy might be important; and, as the engineer figuring on water-power possibilities is usually more concerned with the individual year than with the average for a great many years, a discrepancy of 8 or 10 in. in the estimated run-off would often lead to serious consequences.

* "The Control of Water."

Mr. Link.

Of the author's diagrams, the two in which the observed data most nearly agree with the formula, are those for the Esopus and Schoharie water-sheds. In one case the observations cover only 5 years, and in the other only 4 years, and it might be suggested that the observed data in these cases are too few to furnish a satisfactory basis of comparison with a general formula.

As an illustration of the erratic nature of the relation between rainfall and run-off in some sections of the country, attention is called to a water-shed in California with an area of 250 sq. miles, on which the precipitation and run-off records have been kept with a considerable degree of accuracy. These data are shown in Table 6.

TABLE 6.

Years.	Annual precipitation, in inches.	Total run-off, in inches.	Run-off. Percentage of precipitation.
1906	32.33	3.72	11.54
1907	15.68	2.13	13.50
1908	18.69	0.81	4.34
1909	28.76	1.82	6.33
1910	12.61	1.06	8.40
1911	15.23	0.30	1.97

By an inspection of these data it will be seen that in 1907 there was the highest percentage of run-off, although the precipitation was less than one-half of that of the preceding year; and though the precipitation in 1911 was only slightly below that of 1907, the run-off was only one-seventh as great. Other water-sheds could be cited on which similar conditions obtain, but this one example will be sufficient to illustrate the point.

Mr. Justin lays particular stress on slope and mean annual temperature as the chief factors in determining the manner in which the relation between rainfall and run-off varies from one water-shed to another. The writer, however, would suggest that the geological formation and the distribution of rainfall throughout the year are factors of equal importance, and that if the relation of rainfall to run-off for different water-sheds, or for different years on the same water-shed, is ever reduced to an exact formula, such formula will have to take into account the distribution of rainfall throughout the year, as well as the other factors mentioned.

Mr. Finch.

J. K. Finch, Jun. Am. Soc. C. E. (by letter).—The writer simply desires to point out that the error involved in the method of estimating run-off from rainfall proposed by Mr. Justin is a function of the total time interval considered. Thus, if the formula be applied to the prob-

lem of determining the total run-off for several years from the rainfall for the corresponding period, the resulting error is small and may indeed be accounted for in many cases by the probable error in the rainfall data. If, on the other hand, the formula is applied to individual years, the resulting error is larger because the ground-water conditions may not be the same at the beginning and end of the year, and, when applied to individual months, the resulting run-off is, of course, in many cases absolutely different from the actual. Mr. Finch.

Now, the period to be considered, in estimating the storage for a given draft, is the length of time during which the reservoirs will be below high-water level. This would vary, with different localities, from a few months to such extremes as that mentioned by Mr. Le Conte for conditions near San Francisco, where 3 years were necessary; and on any water-shed it depends on the draft desired and the variations in run-off. Taking Mr. FitzGerald's* figures for the Sudbury watershed, for example, it is found that the period of depletion for drafts of from 200 000 to 650 000 gal. per sq. mile per day is 7½ to 10½ months, which approximates the average period during which the ground-water would be low. For 700 000 gal. and more, the period is 2 years and more. It should also be remembered that these figures are for the years of greatest drought, and that in the average year the periods would be much shorter.

One would expect, therefore, to obtain very close figures in the North Atlantic States for the storage required to maintain a certain draft by Mr. Justin's method, provided this draft was more than 700 000 gal. per sq. mile per day. For drafts of from 650 000 to 200 000 gal. one would find errors of increasing amount and for less than 200 000 gal., as the period of depletion is only a few months, the errors involved would be far in excess of the "20 to 30% in excess of the depletion shown by the mass-curve," which Mr. Justin states it is customary to use. In the average year the results would be in error by still greater quantities, although the average year is not the critical one for storage, and this feature is therefore not important.

The conclusion is, therefore, that Mr. Justin's method is reasonably satisfactory in the North Atlantic States for a few exceptional cases only, that is, where the storage possibilities of a water-shed have been developed to a great extent by building reservoirs, and a correspondingly large draft is to be expected. In the majority of cases economy does not justify such development, and, if the water is to be used for water-supply purposes, it is considered best to limit the period of depletion, and the drafts will generally be less than 200 000 gal. per sq. mile per day. For the solution of the average problem, therefore, the use of this method will be dangerous.

* *Transactions*, Am. Soc. C. E., 1892, Vol. XXVII, p. 268.

Mr. Finch. In these cases, where only a few months' storage is necessary, the method used must take into account ground-storage conditions, and, as far as the writer knows, Mr. Vermeule's method* gives the nearest approach to true run-off figures of any proposed thus far. Mr. Justin's general formula, introducing as it does the hitherto neglected effect of slope, combined with Mr. Vermeule's ground-storage curves, should give a very close approximation for estimating run-off from rainfall month by month.

Mr. Justin. JOEL D. JUSTIN, ASSOC. M. AM. SOC. C. E. (by letter).—That great caution is necessary in any attempt to predict the quantity of run-off from a water-shed, all engineers are agreed. That it is preferable to base calculations on the actual run-off, rather than on estimates founded on the rainfall of the water-shed is almost axiomatic. The methods and formulas presented in the paper are intended for use on water-sheds where run-off data are partly or totally lacking.

Given such a water-shed, one of two courses is open to the engineer desiring to build a storage reservoir: First, he may make a guess as to the run-off, taking it as a percentage of the rainfall, and being guided as much as possible by his judgment. Second, he may make a careful study of the conditions governing the relations of run-off to rainfall on the water-shed and on others having the same general characteristics, and, as the result of these studies, arrive at a run-off on which he may safely rely.

The formula derived by the writer is an attempt to reduce the second method to mathematical terms. Caution and judgment are necessary, of course, in using the formula, just as they are necessary in the use of formulas for, say, the design of a masonry dam. For instance, the writer would not attempt to apply the formula to the streams of California or Kansas without the possession of additional data.

Mr. Le Conte states that the formula gives the long average run-off. This is incorrect. The formula gives equally well the run-off for any particular year. In order to prove this, Tables 4 and 5 were prepared, showing that, in nearly all years, the agreement between the run-off computed by the formula and that recorded was remarkably close. It was even shown that usable mass-curves could be built up by the use of the formula. Mr. Le Conte points to a case in California where for 3 successive years there was no run-off. It is probable that the formula, if applied to this water-shed, would for these 3 years give such a small run-off as to be negligible.

Even on water-sheds where run-off data are available, it is necessary to use them with caution, as measurements of flow may be very inaccurate. The whole subject is one which does not permit of precision.

* Geological Survey of New Jersey, Vol. III, 1894.

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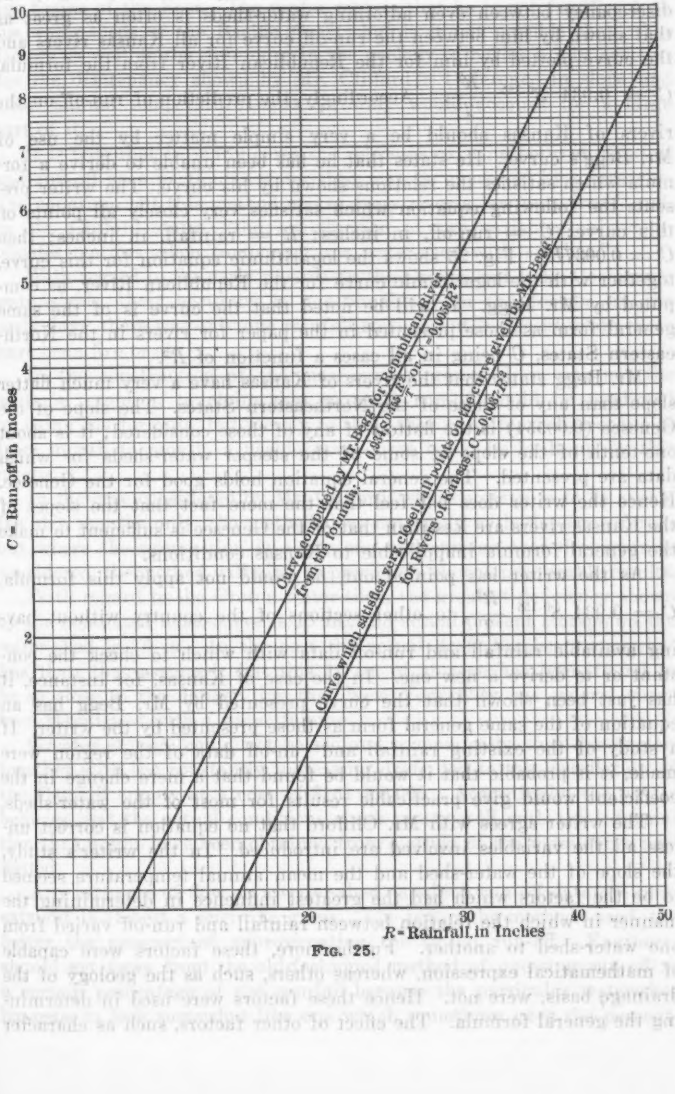

FIG. 25.

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On Fig. 21 Mr. Begg gives a run-off curve for the rivers of Kansas, and states that nearly all points fall on or near this curve. This is a condition which is truly remarkable. In other parts of the country the discrepancy between even adjoining water-sheds is often as great as that shown by him between the run-off curve for all Kansas rivers and the curve plotted by him for the Republican River from the formula, $C = 0.934\ S^{0.155}\ \frac{R^2}{T}$. Accordingly, the prediction of run-off on the rivers of Kansas should be a very simple matter by the use of Mr. Begg's curve. He states that he has been unable to derive a formula which satisfies the relations shown by his curve. The writer presents the following equation which satisfies very closely all points on this curve: C = run-off, in inches; R = rainfall, in inches; then $C = 0.0037R^2$. Fig. 25 shows the logarithmic equation for this curve, together with the logarithmic curve for the Republican River, as computed by Mr. Begg. It will be noted that the curve is of the same general form as those presented in the paper for rivers in the Northeastern States, C being in all cases a function of R^2.

Mr. Begg states that the rivers of Kansas have a very much flatter slope than any of those of the Northeastern States. The slope of the Genesee (0.00554) is the flattest of any of those considered; it is about one-tenth of the slope of some of the steeper water-sheds for which data are presented. The general equation holds good for the Genesee. Hence the writer does not feel that the mere fact that the slopes of the Kansas rivers are less than that of the Genesee is sufficient to make the general formula inapplicable to Kansas conditions.

As the writer has pointed out, he would not apply this formula, $C = 0.934\ S^{0.155}\ \frac{R^2}{T}$, to other sections of the country without having available rainfall and run-off data with which to check the constant or to derive a new one. In the case of Kansas, for instance, it has just been shown that the curve presented by Mr. Begg has an equation of the same general form as those presented by the writer. If a study of the existing rainfall and run-off data of the region were made, it is probable that it would be found that a mere change in the coefficient would give practicable results for most of the water-sheds.

The writer agrees with Mr. Clifford that no equation is correct unless all the variables involved are introduced. In the writer's study, the slope of the water-shed and the mean annual temperature seemed to be the factors which had the greatest influence in determining the manner in which the relation between rainfall and run-off varied from one water-shed to another. Furthermore, these factors were capable of mathematical expression, whereas others, such as the geology of the drainage basis, were not. Hence these factors were used in determining the general formula. The effect of other factors, such as character

of vegetation, extent of forest covering, prevailing winds, relative humidity, barometric pressure, etc., seems to be well within the limit of accuracy of the existing data. At least this is the case with the drainage basins of the East. The writer regrets that he is not familiar with the water-sheds of the Pacific Coast; but is it not possible that other factors as well as the difference in the geology of basins might account for the discrepancy shown by Mr. Clifford? Difference in slope and mean annual temperature most certainly would account for a large part of it. Mr. Justin.

It is true that in many cases the relations existing between rainfall and run-off on a particular water-shed may be expressed by a straight-line equation, but this method makes every river a law unto itself, and, as Mr. Clifford states, "the introduction of two variables only complicates matters when comparing one water-shed with another."

With regard to the early records on the Croton and Lake Cochituate water-sheds, their accuracy is questionable. On the former, in the early days, there was only one rainfall station, but in recent years there have been five or six.

In the construction of the mass-curves presented, it was not the writer's intention to claim that monthly run-off could be predicted by using his formula, but merely to show that in spite of discrepancies from month to month, a usable mass-curve could be built up by this means.

In the case considered by Mr. Clifford, where storage is to be over a few dry months only, the method does not apply with the same force; but where the proposition is a big one and storage over dry years is considered, the method will give, as previously shown, practicable results.

With regard to the Croton water-shed, the greatest deficiency shown by the computed mass-curve is 18 in., the actual present storage on the water-shed is 16 in. In the past the actual draft has been somewhat less than that assumed; but this 16 in. of storage has been found to be so close to the danger line that New York City has several times been threatened with a water famine.

Greater storage could not be secured economically, and this factor, together with the constantly increasing draft, led to the installation of the Catskill supply at a cost of $161 000 000. Accordingly, the writer believes that the actual conditions check pretty well with the information that might have been obtained from a study of the mass-curve presented.

Any estimate of flow on a water-shed on which run-off data are non-existent is at best a scientific guess. The better the methods used, the better the prospect of approximating the actual run-off. Engineers should get away from the old-fashioned method of guessing run-off as a certain percentage of the rainfall because the particular water-shed happens to look somewhat like one which sometimes gave the percent-

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age of run-off guessed at. They should attempt to predict run-off by a careful study of all the data available from surrounding water-sheds. It was as an aid in such a study that the present formula was derived.

Mr. Horton states that the writer's formula is certainly not capable of general application to small drainage areas. On what facts does he base this conclusion? In the paper it was shown that the formula applied equally well to the Connecticut, with its 10 000 sq. miles of water-shed, and to the Sudbury, with its water-shed of 75 sq. miles. What is there in a water-shed containing from 15 to 25 sq. miles to make it inherently different from one containing from 70 to 10 000 sq. miles? Why should water-sheds be divided into two classes: those containing more and those containing less than 25 sq. miles?

The writer regrets that he did not have available more data on the slope, mean annual temperature, rainfall, and run-off of small water-sheds. He is confident, however, that, given equally accurate data on slope, temperature, and rainfall, run-off can be predicted by the formula just as accurately for a small water-shed as for a large one. The difficulty is in obtaining these data. The slope is easily ascertained, but the nearest station having rainfall and temperature records may be many miles distant from the small water-shed in question. If such data are used, and especially if they are used without some logical method of correction, the formula, of course, may give very misleading results.

Mr. Horton cites a water-shed of 0.4 sq. mile, which had a considerable minimum run-off, though an adjoining water-shed of 1.18 sq. miles had a minimum flow of zero. The same thing happens on larger water-sheds, and must be looked out for in the application of any formula. For instance, the writer knows of a stream, which, when measured at a gauging station, where the bottom was ledge rock, gave considerably higher results in midsummer than when measured at another gauging station only a short distance away, where the bottom was a gravel deposit of considerable depth and extent. This matter of water-shed leakage cannot be covered by any formula; but it should not make any trouble for the engineer who applies the formula to a given water-shed, after having made a study of the existing conditions.

As regards cultural and soil conditions, the writer believes that the slope is a fair, if rough, index to these conditions. Thus, as Mr. Horton states, the flat water-shed is pretty sure to consist largely of cultivated lands and meadow with a deep soil covering; and a very steep water-shed is pretty sure to be quite heavily forested and have a light soil covering.

The criticism made by Mr. Horton of the writer's simple method of obtaining slope is merited, from a mathematical standpoint, and he freely acknowledges that the method suggested by Mr. Horton would give a truer index of the slope. Of course, it is a matter of opinion

as to whether or not excessive labor is involved in Mr. Horton's method of deriving slope. The writer believes that most engineers would consider excessive this labor of measuring all the contours on a watershed and obtaining the average distance between them. On several water-sheds the writer has tried to obtain a value for average slope by weighting the average slope between contours according to the area between them, and has found the work very great; and, as it led to no increase in final accuracy, he determined that, even if he did adopt such a method, probably no one else would ever do so. Mr. Horton's method is the more logical, but unless he can show that it would lead to a more accurate prediction of run-off, it would be a mere waste of time to use it. Mr. Justin.

Mr. Horton points out several ideal cases in which the slope indicated by the writer's method would differ from the true slope by from 13 to 75 per cent. If Mr. Horton were considering actual watersheds, the writer believes that he would find it difficult to discover a deviation as great as that represented by the second figure. However, even if this deviation were large, it would not be material, for it requires a change of about 100% in slope to make any material change in the relations of rainfall to run-off.

Just how unimportant Mr. Horton's objection to the writer's method of obtaining slope is, may be shown by the following illustration: In Table 1, the mean annual temperature for the Hudson is 41.9 and for the Connecticut 42, but the slope of the Hudson is 0.0149 and that of the Connecticut 0.0117, a difference of more than 20 per cent. In the table, the values of K for these two water-sheds are, respectively, 0.0118 and 0.0117. In other words, in an actual case, where other conditions remain the same, a difference in slope of 20% causes a difference in computed run-off of less than 1 per cent. The effect of slope is not felt until the change becomes comparatively large. The slopes of the steepest water-sheds considered in the paper are ten times as great as those of the flattest.

Mr. Finch points out that, in the ordinary storage proposition, storage for a few months only is considered, and he believes that it would be dangerous to apply to such a case the writer's method of building up a mass-curve by the formula.

That the method will give more accurate results where storage is to cover the greater part of one or more years, the writer believes, but that it would be dangerous to apply it to a case where only a few months' storage is desired, he does not believe, provided, of course, that due caution and good judgment are used.

If there are no run-off data, some method of determining the size of the reservoir must be found, and the writer believes that, even for brief periods of storage, this method will give more reliable results than can otherwise be obtained.

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Mr. Link objects to the use of the short term of the Esopus and Schoharie records in obtaining the general formula. The record on the Esopus, because of the extreme accuracy with which it was obtained, is entitled to equal or even greater weight with the long-term and far less accurate records on the Croton and Sudbury. On the Esopus as many as 13 rainfall stations were maintained on a watershed of 240 sq. miles, and the run-off was measured with a specially constructed concrete weir. On part of the Sudbury and Croton records, the rainfall was obtained from a single station; later, other stations were added. The discrepancies pointed to in the Sudbury records are the only ones of large magnitude in the entire paper, and are very probably due to the condition stated.

With regard to Mr. Link's statement that a discrepancy of 1 or 2 in. is material, it may be said that the present methods of measurement of rainfall do not justify the expectation of anything closer in the average case. Frequently, on large water-sheds, the average is one rainfall station to 1 000 sq. miles.

The behavior of the California water-shed cited by Mr. Link is certainly very erratic; but a large part of the discrepancy may be explained, perhaps, by variations in the ground-water storage. If Mr. Link had been able to divide his periods into water years, at the end of each of which the ground-water level was practically the same, probably it would be found that periods having the same precipitation would also have practically the same run-off. It would also be worth while to look into the accuracy of the precipitation and run-off data in a case of this kind.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1289

PROGRESS REPORT OF THE SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE.*

I. Introduction.

1.—Appointment and Work of Committee.

In 1903 and 1904 Special Committees were appointed by the American Society of Civil Engineers, American Society for Testing Materials, American Railway Engineering and Maintenance of Way Association, and the Association of American Portland Cement Manufacturers, for the purpose of investigating current practice and providing definite information concerning the properties of concrete and reinforced concrete and to recommend necessary factors and formulas required in the design of structures in which these materials are used. The history of the appointment of the committees is as follows:

At the Annual Convention of the American Society of Civil Engineers, held at Asheville, N. C., June 11th, 1903, the following resolution was adopted:

"It is the sense of this meeting that a Special Committee be appointed to take up the question of concrete and steel-concrete, and that such committee co-operate with the American Society for Testing Materials, and the American Railway Engineering and Maintenance of Way Association."

Following the adoption of this resolution, a Special Committee on Concrete and Steel-concrete was appointed by the Board of Direction on May 31st, 1904. At the Annual Meeting, held January 18th, 1905, the title of this Special Committee was, at the request of the

*Presented to the Annual Meeting, January 15th, 1913.

Committee, changed to "Special Committee on Concrete and Reinforced Concrete." This Special Committee held its first meeting at Atlantic City, N. J., June 17th, 1904, and effected an organization; Mr. C. C. Schneider was appointed Chairman and Mr. J. W. Schaub, Secretary. Mr. Schneider resigned from the Committee on January 3d, 1911, and the Board of Direction, on January 31st, 1911, appointed Mr. J. R. Worcester as Chairman. On the resignation of Mr. J. W. Schaub, Mr. Richard L. Humphrey was appointed Secretary on October 11th, 1905.

At the first meeting of the Committee it was decided to co-operate with similar committees which had been appointed by the American Society for Testing Materials and the American Railway Engineering and Maintenance of Way Association through the organization of a Joint Committee on Concrete and Reinforced Concrete.

Subsequent meetings of the Committee were held at New York in January, 1906, 1907, 1909, 1910, 1911, and 1912; on December 10th, 1907; on March 1st, April 3d, May 1st, 1911, and on November 20th, 1912; at Philadelphia in December, 1908; and at Chicago in June, 1910. The Committee was also represented at all the meetings of the Joint Committee.

At the annual meeting of the American Society for Testing Materials, held on July 1st, 1903, at the Delaware Water Gap, the following resolution was unanimously adopted:

"That the Executive Committee be requested to consider the desirability of appointing a committee on 'Reinforced Concrete', with a view of co-operating with the committees of other societies in the study of the subject."

At the meeting of the Executive Committee of the American Society for Testing Materials, held on December 5th, 1903, a special committee on "Reinforced Concrete" was appointed.

The American Railway Engineering and Maintenance of Way Association appointed a Committee on Masonry on July 20th, 1899, with instructions, as a part of its duties, to prepare specifications for concrete masonry. A preliminary set of specifications for Portland cement concrete was reported to and adopted by the Association on March 19th, 1903. At the meeting held in Chicago on March 17th, 1904, the Committee on Masonry was authorized to co-operate with the Special Committee on Concrete and Reinforced Concrete of the American Society of Civil Engineers, and, following this action, a special sub-committee was appointed.

At a meeting of the several special committees representing the above-mentioned societies, held at Atlantic City, N. J., on June 17th, 1904, arrangements were completed for collaborating the work of these several committees through the formation of the Joint Committee on

Concrete and Reinforced Concrete. Mr. C. C. Schneider was elected temporary chairman and Professor A. N. Talbot temporary secretary. The proposed plan of action of the Special Committee of the American Society of Civil Engineers was outlined, involving the appointment of sub-committees on Plan and Scope, on Tests, and on Ways and Means.

The Joint Committee, at its first meeting, invited the Association of American Portland Cement Manufacturers to join in its deliberations through a committee appointed for the purpose.

The Joint Committee, at meetings at St. Louis in October, 1904, and at New York in the following January, perfected its organization by the adoption of rules and the choice of Mr. C. C. Schneider as Chairman, Mr. Emil Swensson, Vice-Chairman, and Mr. J. W. Schaub, Secretary. Later, on the resignation of Mr. Schaub, Mr. Richard L. Humphrey was chosen Secretary. Sub-committees on Plan and Scope, on Tests, and on Ways and Means were appointed.

The Joint Committee, as thus organized, consisted of the following members:

OFFICERS.

Chairman.—C. C. SCHNEIDER.
Vice-Chairman.—EMIL SWENSSON.
Secretary.—RICHARD L. HUMPHREY.

MEMBERS.

American Society of Civil Engineers (Special Committee on Concrete and Reinforced Concrete):

Greiner, J. E., Consulting Engineer, Baltimore and Ohio Railroad, Baltimore, Md.
Hatt, W. K., Professor of Civil Engineering, Purdue University, Lafayette, Ind.
Hoff, Olaf, Vice-President, Butler Brothers, Hoff and Company, New York, N. Y.
Humphrey, Richard L., Consulting Engineer; Engineer in Charge, Structural Materials Testing Laboratories, U. S. Geological Survey, Philadelphia, Pa.
Lesley, R. W., President, American Cement Company, Philadelphia, Pa.
Schaub, J. W., Consulting Engineer, Chicago, Ill.
Schneider, C. C., Consulting Engineer, Philadelphia, Pa.
Swensson, Emil, Consulting Engineer, Pittsburgh, Pa.
Talbot, A. N., Professor of Municipal and Sanitary Engineering, in Charge of Theoretical and Applied Mechanics, University of Illinois, Urbana, Ill.
Worcester, J. R., Consulting Engineer, Boston, Mass.

American Society for Testing Materials (Committee on Reinforced Concrete):

Fuller, William B., Consulting Engineer, New York, N. Y.

Heidenreich, E. Lee, Consulting Engineer, New York, N. Y.

Humphrey, Richard L., Consulting Engineer; Engineer in Charge, Structural Materials Testing Laboratories, U. S. Geological Survey, Philadelphia, Pa.

Johnson, Albert L., Consulting Engineer, St. Louis, Mo.

Lanza, Gaetano, Professor of Theoretical and Applied Mechanics, Massachusetts Institute of Technology, Boston, Mass.

Lesley, R. W., President, American Cement Company, Philadelphia, Pa.

Marburg, Edgar, Professor of Civil Engineering, University of Pennsylvania, Philadelphia, Pa.

Mills, Charles M., Principal Assistant Engineer, Philadelphia Rapid Transit Company, Philadelphia, Pa.

Moisseiff, Leon S., Engineer of Design, Department of Bridges, New York, N Y.

Quimby, Henry H., Assistant Engineer of Bridges, Bureau of Surveys, Philadelphia, Pa.

Taylor, W. P., Engineer in Charge of Testing Laboratory, Philadelphia, Pa.

Thompson, Sanford E., Consulting Engineer, Newton Highlands, Mass.

Turneaure, F. E., Dean of College of Mechanics and Engineering, University of Wisconsin, Madison, Wis.

Wagner, Samuel Tobias, Assistant Engineer, Philadelphia and Reading Railroad, Philadelphia, Pa.

Webster, George S., Chief Engineer, Bureau of Surveys, Philadelphia, Pa.

American Railway Engineering Association (Sub-Committee on Reinforced Concrete):

Beckwith, Frank, Engineer of Bridges and Structures, Lake Shore and Michigan Southern Railroad, Cleveland, Ohio.

Boynton, C. W., Inspecting Engineer, Cement Department, Illinois Steel Company, Chicago, Ill.

Cunningham, A. O., Chief Engineer, Wabash Railroad, St. Louis, Mo.

Scribner, Gilbert H., Jr., Contracting Engineer, Chicago, Ill.

Swain, George F., Professor of Civil Engineering, Massachusetts Institute of Technology, Boston, Mass.

Association of American Portland Cement Manufacturers (Committee on Concrete and Steel Concrete):

Fraser, Norman D., President, Chicago Portland Cement Company, Chicago, Ill.

Griffiths, R. E., Vice-President, American Cement Company, Philadelphia, Pa.

Hagar, Edward M., Manager, Cement Department, Illinois Steel Company, Chicago, Ill.

Newberry, Spencer B., Manager, Sandusky Portland Cement Company, Sandusky, Ohio.

Since organization, the following changes have occurred in the *personnel* of the Joint Committee:

J. W. Schaub, died March 30th, 1909.
C. C. Schneider, resigned January 3d, 1911.
Ernest R. Ackerman, resigned.
T. J. Brady, resigned.
Frank Beckwith, resigned.
A. O. Cunningham, resigned.
George F. Swain, resigned.

The following representatives of the American Railway Engineering Association have since been appointed:

Thompson, F. L., Engineer of Bridges and Buildings, Illinois Central Railroad, Chicago, Ill.

Alternates:

Hotchkiss, L. J., Assistant Bridge Engineer, Chicago, Burlington and Quincy Railroad, Chicago, Ill.

Prior, J. H., Assistant Engineer, Chicago, Milwaukee and St. Paul Railway, Chicago, Ill.

Schall, F. E., Bridge Engineer, Lehigh Valley Railroad, South Bethlehem, Pa.

Tuthill, Job, Assistant Engineer, Cincinnati, Hamilton and Dayton Railway, Cincinnati, Ohio.

At a meeting of the Joint Committee held at Atlantic City, N. J., on June 30th, 1911, Mr. J. R. Worcester was elected Chairman.

Meetings of the Joint Committee have been held as follows:

June 17th, 1904, at Atlantic City, N. J.
Oct. 4th, 5th, 6th, 1904, at St. Louis, Mo.
Jan. 17th, 1905, at New York, N. Y.
June 21st, 1905, at Cleveland, Ohio.
June 30th, 1905, at Atlantic City, N. J.
Oct. 11th, 1905, at New York, N. Y.

Dec. 14th, 1905, at New York, N. Y.
June 21st, 1906, at Atlantic City, N. J.
Dec. 13th, 1906, at New York, N. Y.
Jan. 15th, 1907, at New York, N. Y.
March 7th, 1907, at New York, N. Y.
March 18th, 1907, at Chicago, Ill.
June 21st, 22d, 1907, at Atlantic City, N. J.
Dec. 10th, 1907, at New York, N. Y.
Oct. 27th, 28th, 1908, at New York, N. Y.
Dec. 9th, 10th, 11th, 1908, at Philadelphia, Pa.
June 30th, 1911, at Atlantic City, N. J.
Nov. 20th, 1912, at New York, N. Y.

At the meeting of the Joint Committee at St. Louis in October, 1904, it was determined to arrange tests at such technological institutions as were provided with the requisite facilities and were willing to co-operate, the Committee, through its sub-committee on Ways and Means, to provide materials, and through its sub-committee on Tests, to consult as to lines of testing and to advise as to methods. The following ten institutions, Case School of Applied Science, Columbia University, Cornell University, University of Illinois, State University of Iowa, Massachusetts Institute of Technology, University of Minnesota, Ohio State University, Purdue University, and University of Wisconsin, undertook a preliminary series of tests, and carried them through, in due time reporting their results to the Committee.

Through the inability of the Committee to do as much as it had hoped by way of furnishing uniform materials for these tests and exercising a proper supervision, the results were not as serviceable as they would have been if the full plans had been carefully carried out; but much important information was received in this manner, and the Committee desires to express its gratitude to the professors and students who assisted so kindly in this work.

The results were collated and edited by the Secretary of the Committee at the Structural Materials Testing Laboratories of the U. S. Geological Survey, St. Louis, and the results, in typewritten form, were circulated among the members of the Committee. It was hoped that they might be published by the Geological Survey as a Bulletin, but in that the Committee was disappointed, though some of the results have been published in bulletins and papers issued by their authors.

In June, 1905, the U. S. Geological Survey proposed to co-operate with the Joint Committee to the extent of placing the tests made at the St. Louis Laboratory at its service and allowing the Committee the privilege of advising as to what tests of concrete and reinforced concrete should be conducted there. This co-operation was welcomed by the Committee, and was brought about by the fact that the Secretary

of the Committee, who was also the Chairman of the sub-committee on tests, was in charge of the St. Louis Laboratory.

During the five years in which the investigations of structural materials were in progress under the direction of the United States Geological Survey, a large amount of data relating to concrete and reinforced concrete was obtained. These investigations have included the survey of the constituent materials of concrete, such as sands, gravels, and crushed stone, in the various parts of the United States, covering their strength as mortars or concretes in various consistencies and proportions.

A number of series of tests of plain and reinforced concrete beams was made, covering the influence of character of aggregates, proportions and age, percentage of reinforcement, the effect of the variation in span relative to the depth, methods of anchorage of the reinforcement, etc., upon strength. A study was made of the effect of the personal equation in tests of beams, made by three construction companies operating in St. Louis, and by the employees of the testing laboratory. Tests covering bond, shear, compressive strength, and weight per cubic foot, for various classes of aggregates, were made.

Among other investigations were tests of reinforced concrete slabs of 12 ft. span, supported at two and four edges, strength and other properties of cement hollow building blocks, of the permeability of cement mortars and concretes, value of various water-proofing and damp-proofing preparations, effect of alkali and sea water on cement mortars and concretes, the fire-resistive properties of concrete and other structural materials, and these have been made and published, in part.

The collation and study of the data obtained were seriously handicapped through lack of funds available for this purpose, the large part of the appropriation being devoted to work urgently required by the various Government Bureaus. Of the annual Government appropriation of \$100 000, there was never available more than \$15 000 per annum for the investigation of concrete and reinforced concrete, and for several years the amount did not exceed \$5 000 a year. None of this was available for the publication of results, and the allotment from the funds provided for all Government printing was wholly inadequate for the purpose.

On June 30th, 1910, Congress transferred this work to the Bureau of Standards, together with the data collected. It is understood that arrangements have been made by which the data of the tests will be published as rapidly as conditions permit.

The Committee has had the benefit of the results of investigations by a number of laboratories, some of which were under the direct supervision of its members. The extent and varied character of the

tests, and their interpretation by those in charge, made them of especial value to the Committee.

The Committee also has had the advantage of investigations made in foreign laboratories.

At a meeting of the Joint Committee at Atlantic City, on June 30th, 1905, it was decided to divide among its members the work of collating and digesting the results of all available tests on concrete and reinforced concrete, and, in pursuance of this resolution, sub-committees were appointed on the following subjects:

Historical.
Aggregates, Proportions and Mixing.
Physical Characteristics, Water-proofing, etc.
Strength and Elastic Properties.
Simple Reinforced Concrete Beams.
T-Beams, Floor Slabs, etc.
Columns and Piles.
Fire-resistive Qualities.
Failures of Concrete Structures.
Arches.

A large amount of work was done by these sub-committees, and extensive reports were submitted by most of them. These reports were typewritten in manifold and circulated among the members of the Joint Committee, and were of great value to the Committee in arriving at its conclusions.

The Sub-Committee on Ways and Means raised by subscription about $8 000, which was used for preliminary investigations and expenses incident to printing its report and carrying on the work of the Committee. The Committee desires to express its appreciation for contributions and for donations of materials.

Even with this support, the field of activity of the Committee has been limited in scope, and it has been unable to undertake investigations of its own.

In 1908 the Committee began the preparation of the Progress Report which was submitted to the Society in January, 1909. A preliminary outline was prepared by the Secretary and submitted to the Committee in October. On October 27th, a meeting of the Joint Committee was held at New York, at which the report was discussed paragraph by paragraph, and chapters were referred to sub-committees and carefully revised during the following three weeks. The whole, as thus amended and revised, was again submitted in print to a full meeting held at Philadelphia, on December 9th, 10th, and 11th, and again was gone over in great detail. As a result of those two meetings, a considerable amount of matter, which it was at first intended to include, was omitted on account of slight disagreements as to its form

and lack of time to work it into satisfactory shape, and to this fact may be attributed some of the criticisms which have been elicited. It is hoped, in this report, to avoid these objections.

In the spring of 1911 the work of revising the 1909 Progress Report was taken up, and a number of meetings were held. The discussions submitted to the American Society of Civil Engineers and subsequent papers relating to the same subject were carefully considered, and differences of opinion between members of the Committee were threshed out.

Through the co-operation of the societies represented on the Joint Committee, the report was again put in type, and the necessary editions were printed for the use of the members of the Committee, the last bearing the date of August 1st, 1911. In the form thus reached, the report remained until November 20th, 1912, when the Committee again met in New York and gave the final review needed to bring it into the shape in which it is now presented.

2.—Historical Sketch of Use of Concrete and Reinforced Concrete.

In considering the history of concrete and reinforced concrete, a distinction should be made between the two. The use of concrete extends back to long before the Christian era—on the other hand, the art of reinforced concrete is in its infancy.

The use of concrete by the ancient Romans was due to the discovery of the fact that volcanic ash or puzzolan, when mixed with slaked lime, made a cement possessing hydraulic properties. The durability of this work of the Romans was due largely to favorable climatic conditions and the character of the cement used.

From the downfall of the Roman Empire to the last half of the Eighteenth Century the manufacture of cement seems to have been discontinued. The Roman cement mortars and concretes surviving the ravages of the elements became so hard that the cement acquired a reputation that led the early experimenters of the Eighteenth Century to seek to recover this supposedly lost Roman art. Evidently, no concrete was used during this period, for the necessity of simultaneous induration in the interior and exterior of the mass prevents the use of lime alone in concrete, and requires the use of some material having hydraulic qualities. This fact limited the use of concrete to regions where hydraulic limes and cements were to be found.

In 1756 Smeaton discovered that an argillaceous limestone produced a lime that would set and harden under water, but no immediate appreciation of this knowledge appears to have resulted.

Natural cement was first manufactured by Parker in 1795, as a result of an attempt to equal or excel Roman cement, and in 1796 he took out an English patent. Natural cement was first produced in America in 1818, and for a long time was the principal cement used.

With the introduction of Portland cement, and the reduction in the cost of manufacture, there has been a gradual substitution of Portland for natural cement. The production of natural cement reached a maximum of nearly 10 000 000 bbl. in 1899 and gradually decreased to about 9 000 000 bbl. in 1911.

The art of manufacturing Portland cement was discovered in 1811 by Joseph Aspdin, and patented by him in 1824. He called this cement "Portland" by reason of its resemblance to a building stone obtained from the Isle of Portland, off the coast of England. Up to 1850 very little progress was made in the manufacture of this cement in England. Since 1855, however, the increase in the production in Europe has been steady, and its superiority has led to a gradually increasing use in such structures as require concrete in mass, as foundations, fortifications, sea-walls, docks, locks, etc. While Portland cement was first manufactured in 1824, and was produced in 1871 by David O. Saylor, at Coplay, Pa., and by Thomas Millen, at South Bend, Ind., it was not until the early Eighties that it was manufactured to any extent in America. From that time on, the production has rapidly increased, reaching the enormous total of nearly 80 000 000 bbl. in 1911. This increase in production has been largely stimulated by the reduction in cost of Portland cement through the perfection of the American methods, the introduction of reinforced concrete, and the extensive use of cement during the last few years.

In 1850 Joseph Gibbs obtained a British patent for casting solid walls in wooden moulds, and in 1897 C. W. Stevens obtained a patent for making artificial cast stone with concrete. It is not clear, however, that these inventors were the first to use the material in a similar way.

The origin of the idea of increasing the load-carrying capacity of concrete by reinforcing it with metal embedded in it is generally attributed to Joseph Monier, a French gardener, who used a wire frame or skeleton embedded in concrete in the construction of flower pots, tubs, and tanks, in 1867, and for which he obtained the first patent of the kind in the same year. This was not the first use of the material, however, as Lambot constructed a boat of reinforced concrete in 1850, which was exhibited at the Paris Exposition in 1853. He took out an English patent in 1855.

In France, in 1861, François Coignet applied the principles of reinforced concrete in the construction of beams, arches, pipes, etc., and with Monier exhibited some of their work at the Paris Exposition of 1867. Coignet also took out an English patent in 1855. In England, in 1854, W. B. Wilkinson took out a patent for a reinforced concrete floor. In America, Ernest L. Ransome used metal in combination with concrete as early as 1874, and W. E. Ward erected, in 1875, at Port Chester, New York, a house built entirely of reinforced concrete.

Monier, while not the first to apply it, obtained the first patents for reinforced concrete, the German and American rights of which he disposed of to G. A. Wayss and Company in 1880. Wayss and J. Bauschinger, shortly after, began the tests on this material which were published in 1887.

Thaddeus Hyatt, an American engineer, employed David Kirkaldy of London to make the experiments on reinforced concrete which Hyatt published in 1877. The theories of Hyatt were applied in a practical way to building construction in 1877 by H. P. Jackson, of San Francisco.

In America, Ransome, between 1874 and 1884, constantly increased his application of metal reinforcement consisting of old wire rope and hoop iron, gradually realizing the necessity for using it with a greater regard for its proper position in the mass, and in 1884 took out the first patent for a deformed bar. Prior to this, reinforced concrete was used but little in the United States. Ransome built his first important structure in 1890, the Leland Stanford, Jr., Museum Building, 312 ft. long, two stories high, with basement, the walls and floors of which were of reinforced concrete. Since 1891, when the first slabs of reinforced concrete were used in America, the development has been rapid.

The introduction of this form of construction proceeded more slowly in Europe, and between 1891 and 1894 Moeller in Germany, Wünsch and Emperger in Hungary, Melan in Austria, and Hennebique in France, were pioneers in its development. Hennebique built reinforced concrete slabs as early as 1879, but did not patent his system of construction until 1892.

The first published method of computation was by Koenen and Wayss in 1886. Subsequent theories have been advanced by de Mazas, Neuman, Melan, Coignet, de Tedesco, Von Thullie, Ostenfeld, Sanders, Spitzer, Lutken, Ritter, Hatt, Talbot, Turneaure, and others. As early as 1884, Ransome worked out methods of calculation independent of other investigators, and in 1899 Considère published his important series of tests from which he deduced his methods of calculation.

During the last ten years the earlier theories have been somewhat modified as experience has been gained and as the fund of experimental knowledge has accumulated. The trend of the modifications has been toward greater harmony in methods of calculation. Some of the earlier assumptions have been proved fallacious, and generally abandoned. On the other hand, some of the refinements of calculation, though known to be in accordance with facts, have, by general consent, been discarded, as they do not affect the design materially, or are taken into account by a modification of the constants. Among these are the value of the concrete in the tension side of a beam and the lack of a uniform modulus of elasticity in compression of concrete under widely varying stress. The earlier theories did not deal with the

diagonal tension under shearing stresses. This has been found to be a most important consideration, and much attention has been paid to it in recent years. In spite of the study which has already been given to it, however, there is still much to learn in this direction. The action of various forms of reinforcement in columns has received much consideration, and there is still a wide difference of opinion as to the efficacy of some forms of column reinforcement. Many experiments have been made in this branch of the subject, and practice appears to be gradually converging toward greater uniformity.

In the preparation of this historical sketch, the Committee has endeavored to verify the facts, and has received the co-operation of H. Kempton Dyson, Secretary of the Concrete Institute of England, Alfred Huser, President, Deutscher Beton-Verein, C. von Bach, Otto Leube, of Germany, Karl Naehr, of Austria, Joseph Schustler, of Hungary, and H. I. Hannover, of Denmark, to whom the Committee wishes to acknowledge its appreciation and thanks.

3.—Authorities on Which Recommendations are Based.

It has been suggested that a report such as this should include all the data on which conclusions are based. The impracticability of this may not be realized by those who are not familiar with the enormous quantity of matter involved. There are, however, reasons other than the magnitude of the task which tend to show that full publication is not advisable. One of these is that most of the experimental results have already appeared in print and are now available, and a reprint of them would be of no great advantage to any one. Where originally printed they are frequently accompanied with comments and deductions by their authors, which are of great value as such but could scarcely be copied by the Committee. Another reason against publication is that, in the large part of the experimental work consulted, it has been found that certain vitally important information, either with regard to the materials, the way in which they are manipulated, or as to the precise results reached, is lacking. The omission of measurements of deformations, of course, frequently renders results of little value. While such tests may have some use on account of particular facts developed, a large part may be useless, and consequently unsuitable for publication. The difficulty of separating the valuable from the valueless would be almost insurmountable.

It may not be improper, however, to append the following list of authors and references, as comprising a considerable part of the most important published material on the subject under consideration:

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E. Mörsch.—*"Der Eisenbetonbau."*

Charles L. Norton.—"Fire-proofing, Protection of Steel by Concrete": Boston Insurance Engineering Experiment Station Reports, IV and IX.

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In addition to the authorities above quoted, the Committee desires to acknowledge with thanks the discussions of its progress reports, which have appeared from time to time, and to say that all the points brought out therein have been carefully weighed.

4.—Character of Report Presented.

At the time of the appointment of the Committee, in 1904, there existed a great diversity of opinion in America as to methods of design, safe allowable working stresses, and methods of proportioning, hand-

ling, etc. A great deal of experimental work had been done, but there was need of a clearing house through which results could be compared and divergent views harmonized. During the interval between the appointment of the Committee and the preparation of its first Progress Report rapid advance was made in the art of concrete construction aided by the results of the investigations and the experience acquired by constructors. This report, which was submitted in 1909, attempted to embody recommendations for safe methods of construction and design in accordance with the best practice of the day. It would have been impossible for such a report to meet with the approval of all, and the Committee is well satisfied that its most vital recommendations have met with quite general acceptance by the engineers of the country.

Since the appearance of the first Progress Report, many experiments have been conducted by some of the technical institutions and by private and corporate interests, and through these and through longer experience in construction by its members and others, the Committee is now able to make some perfecting modifications of its former report and to add some entirely new material. The time, therefore, seems opportune for presenting this second report, bringing the work up to date.

The Committee would point out that while the report deals with every kind of stress to which concrete is subjected, and includes all ordinary conditions of proportioning and handling, it does not go into all types of construction or all the applications to which concrete and reinforced concrete may be put.

It is not to be assumed that the Committee in presenting this report wishes to imply that further improvements are not possible. A careful reading will disclose many points on which the present deductions are regarded as only tentative; but it has been the aim of the Committee to cover as fully as possible recommendations based on the present state of the art.

This report is what the word implies, and nothing more; it is not a "specification", but may be used as a basis for specifications.

The use of concrete and reinforced concrete involves the exercise of good judgment to a greater degree than for any other building material.

Rules cannot produce or supersede judgment; on the contrary, judgment should control the interpretation and application of rules.

II. Adaptability of Concrete and Reinforced Concrete.

The adaptability of concrete and reinforced concrete for engineering structures, or parts thereof, is now so well established that they may be considered the recognized materials of construction. They have proved satisfactory materials, when properly used, for those purposes for which their qualities make them particularly suitable.

1.—Uses.

Concrete is a material of very low tensile strength, and capable of sustaining but very small tensile deformations without rupture; its value as a structural material depends chiefly on its durability, its fire-resistive qualities, its strength in compression, its relatively low cost, and its adaptability to placing, especially where space is cramped or limited. Its strength increases generally with age.

Concrete is well adapted for structures in which the principal stresses are compressive, such as foundations, dams, retaining and other walls, tunnels, piers, abutments, short columns, and, in many cases, arches. In the design of massive concrete, the tensile strength of the material in resisting principal stresses must generally be neglected.

By the use of metal reinforcement to resist the principal tensile stresses, concrete becomes available for general use in a great variety of structures and structural forms. This combination of concrete and metal is particularly advantageous in the beam, where both compression and tension exist; it is also advantageous in the column, where the main stresses are compressive, but where cross-bending may exist. In structures resisting lateral forces, it possesses advantages over plain concrete in that it may be designed so as to utilize more fully the strength rather than the weight of the material. Metal reinforcement may also be of value in distributing cracks due to shrinkage and temperature changes.

2.—Precautions.

Failures of reinforced concrete structures are usually due to any one or a combination of the following causes: defective design, poor material, faulty execution, and premature removal of forms.

The defects in a design may be many and various. The computations and assumptions on which they were based may be faulty and contrary to the established principles of statics and mechanics; the unit stresses used may be excessive, or the details of the design defective.

Articulated concrete structures designed in imitation of steel trusses, may be mentioned as illustrating a questionable use of reinforced concrete, and such structures are not recommended.

Poor material is sometimes used for the concrete, as well as for the reinforcement. The use of poor aggregates, especially sand, which have not been tested, is a common source of defect. Inferior concrete is frequently due also to lack of experience on the part of the contractor and his superintendents, or to the absence of proper supervision.

An unsuitable quality of metal for reinforcement is sometimes prescribed in specifications, for the purpose of reducing the cost. For steel structures, a high grade of material is specified, but the steel used

for reinforcing concrete is sometimes made of unsuitable, brittle material.

Faulty execution, careless workmanship, and too early removal of forms may generally be attributed to unintelligent or insufficient supervision.

3.—Responsibility and Supervision.

The design of reinforced concrete structures should receive at least the same careful consideration as those of steel, and only engineers with sufficient experience and good judgment should be intrusted with such work.

The computations should include all minor details, which are sometimes of the utmost importance. The design should show clearly the size and position of the reinforcement, and should provide for proper connections between the component parts, so that they cannot be displaced. As the connections between reinforced concrete members are frequently a source of weakness, the design should include a detailed study of such connections, accompanied by computations to prove their strength.

While other engineering structures on the safety of which human lives depend are generally designed by engineers employed by the owner, and the contracts let on the engineer's design and specifications, in accordance with legitimate practice, reinforced concrete structures frequently are designed by contractors or by engineers commercially interested, and the contract let for a lump sum.

The construction of buildings in large cities is regulated by ordinances or building laws, and the work is inspected by municipal authorities. For reinforced concrete work, however, the limited supervision which municipal inspectors are able to give is not sufficient. Therefore, means for more adequate supervision and inspection should be provided.

The execution of the work should not be separated from the design, as intelligent supervision and successful execution can be expected only when both functions are combined. The engineer who prepares the design and specifications, therefore, should have the supervision of the execution of the work.

The Committee recommends the following rules for structures of reinforced concrete for the purpose of fixing the responsibility and providing for adequate supervision during construction:

(*a*) Before work is commenced, complete plans shall be prepared, accompanied by specifications, stress computations, and descriptions showing the general arrangement and all details. The plans shall show the size, length, dimensions for points of bending, and exact position of all reinforcement, including stirrups, ties, hooping, and splicing.

The computations shall give the loads assumed separately, such as dead and live loads, wind, and impact, if any, and the resulting stresses.

(*b*) The specifications shall state the qualities of the materials to be used for making the concrete, and the manner in which they are to be proportioned.

(*c*) The strength which the concrete is expected to attain after a definite period shall be stated in the specifications.

(*d*) The drawings and specifications shall be signed by the engineer and the contractor.

(*e*) Plans and specifications for all public structures should be approved by a legally authorized State or City official, and copies of such plans and specifications placed on file in his office.

(*f*) The approval of plans and specifications by other authorities shall not relieve the engineer or the contractor of responsibility.

(*g*) Inspection during construction shall be made by competent inspectors employed by and under the supervision of the engineer, and shall cover the following:

1. The materials.
2. The correct construction and erection of the forms and the supports.
3. The sizes, shapes, and arrangement of the reinforcement.
4. The proportioning, mixing, and placing of the concrete.
5. The strength of the concrete, by tests of standard test pieces made on the work.
6. Whether the concrete is sufficiently hardened before the forms and supports are removed.
7. Prevention of injury to any part of the structure by and after the removal of the forms.
8. Comparison of dimensions of all parts of the finished structure with the plans.

(*h*) Load tests on portions of the finished structure shall be made where there is reasonable suspicion that the work has not been properly performed, or that, through influences of some kind, the strength has been impaired. Loading shall be carried to such a point that one and three-quarters times the calculated working stresses in critical parts are reached, and such loads shall cause no injurious permanent deformations. Load tests shall not be made until after 60 days of hardening.

4.—Destructive Agencies.

(*a*) *Corrosion of Metal Reinforcement.*—Tests and experience indicate that steel sufficiently embedded in good concrete is well protected against corrosion, no matter whether located above or below water level. It is recommended that such protection be not less than 1 in. in thickness. If the concrete is porous, so as to be readily permeable

by water, as when the concrete is laid with a very dry consistency, the metal may corrode on account of the presence of moisture and air.

(*b*) *Electrolysis.*—The most recent experimental data available on this subject seem to show that while reinforced concrete structures may, under certain conditions, be injured by the flow of electric current in either direction between the reinforcing material and the concrete, such injury is generally to be expected only where voltages are considerably higher than those which usually occur in concrete structures in practice. If the iron be positive, trouble may manifest itself by corrosion of the iron accompanied by cracking of the concrete, and, if the iron be negative, there may be a softening of the concrete near the surface of the iron, resulting in a destruction of the bond. The former, or anode effect, decreases much more rapidly than the voltage, and almost if not quite disappears at voltages that are most likely to be encountered in practice. The cathode effect, on the other hand, takes place even on very low voltages, and is therefore more important from a practical standpoint than that of the anode.

Structures containing salt or calcium chloride, even in very small quantities, are very much more susceptible to the effects of electric currents than normal concrete, both the anode and cathode effects progressing much more rapidly in the presence of chlorine.

There is great weight of evidence to show that normal reinforced concrete structures free from salt are in very little danger under most practical conditions, while non-reinforced concrete structures are practically immune from electrolysis troubles.

The results of experiments now in progress may yield more conclusive information on this subject.

(*c*) *Sea Water.*—The data available concerning the effect of sea water on concrete or reinforced concrete are limited and inconclusive. Sea walls out of the range of frost action have been standing for many years without apparent injury. In many harbors where the water is brackish, through rivers discharging into them, serious disintegration has taken place. This has occurred chiefly between low and high tide levels, and is due, evidently, in part to frost. Chemical action also appears to be indicated by the softening of the mortar. To effect the best resistance to sea water, the concrete must be proportioned, mixed, and placed so as to prevent the penetration of sea water into the mass or through the joints. The cement should be of such chemical composition as will best resist the action of sea water; the aggregates should be carefully selected, graded, and proportioned with the cement so as to secure the maximum possible density; the concrete should be thoroughly mixed; the joints between old and new work should be made water-tight; and the concrete should be kept from exposure to sea water until it is thoroughly hard and impervious.

(*d*) *Acids.*—Concrete of first-class quality, thoroughly hardened, is affected appreciably only by strong acids which seriously injure other materials. A substance like manure is injurious to green concrete, but after the concrete has hardened thoroughly it resists the action of such acid satisfactorily.

(*e*) *Oils.*—When concrete is properly made and the surface is carefully finished and hardened, it resists the action of such mineral oils as petroleum and ordinary engine oils. Oils which contain fatty acids produce injurious effects, forming compounds with the lime which result in a disintegration of the concrete in contact with them.

(*f*) *Alkalies.*—The action of alkalies on concrete is problematical. In the reclamation of arid land, where the soil is heavily charged with alkaline salts, it has been found that concrete, stone, brick, iron, and other materials are injured under certain conditions. It would seem that at the level of the ground-water, in an extremely dry atmosphere, such structures are disintegrated, through the rapid crystallization of the alkaline salts, resulting from the alternate wetting and drying of the surface. Such destructive action can be prevented by the use of a protective coating, and is minimized by securing a dense concrete.

III. Materials.

A knowledge of the properties of the materials entering into concrete and reinforced concrete is the first essential. The importance of the quality of the materials used cannot be overestimated, and not only the cement but also the aggregates should be subject to such definite requirements and tests as will insure concrete of the desired quality.

1.—Cement.

There are available for construction purposes: Portland, Natural, and Puzzolan or Slag cements. Only Portland cement is suitable for reinforced concrete.

(*a*) *Portland Cement.*—This is the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials. It has a definite chemical composition varying within comparatively narrow limits.

Portland cement should be used in reinforced concrete construction and any construction that will be subject to shocks or vibrations or stresses other than direct compression.

(*b*) *Natural Cement.*—This is the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas. Although the limestone must have a certain composition, this composition may vary within much wider limits than in the case of Portland cement. Natural cement does not develop its strength as quickly, nor is it as uniform in composition, as Portland cement.

Natural cement may be used in massive masonry where weight rather than strength is the essential feature.

Where economy is the governing factor, a comparison may be made between the use of natural cement and a leaner mixture of Portland cement that will develop the same strength.

(c) *Puzzolan or Slag Cement.*—This is the finely pulverized product resulting from grinding a mechanical mixture of granulated basic blast furnace slag and hydrated lime.

Puzzolan cement is not nearly as strong, uniform, or reliable as Portland or natural cement, is not used extensively, and never in important work; it should be used only for foundation work underground where it is not exposed to air or running water.

(d) *Specifications.*—The cement should meet the requirements of the Standard Methods of Testing and Specifications for Cement (see Appendix, p. 427), or as may be hereafter amended, the result of the joint labors of Special Committees of the American Society of Civil Engineers, American Society for Testing Materials, American Railway Engineering Association, and others.

2.—Aggregates.

Extreme care should be exercised in selecting the aggregates for mortar and concrete, and careful tests made of the materials for the purpose of determining their qualities and the grading necessary to secure maximum density* or a minimum percentage of voids.

(a) *Fine Aggregate.*—This should consist of sand, crushed stone, or gravel screenings, graded from fine to coarse, and passing when dry a screen having holes ¼ in. in diameter; it is preferable that it be of silicious material, and should be clean, coarse, free from dust, soft particles, vegetable loam, or other deleterious matter; and not more than 6% should pass a sieve having 100 meshes per lin. in. Fine aggregates should always be tested.

Fine aggregate should be of such quality that mortar composed of one part Portland cement and three parts fine aggregate by weight, when made into briquettes, will show a tensile strength at least equal to the strength of 1:3 mortar of the same consistency made with the same cement and standard Ottawa sand.† If the aggregate be of poorer quality, the proportion of cement in the mortar should be increased to secure the desired strength.

If the strength developed by the aggregate in the 1:3 mortar is less than 70% of the strength of the Ottawa sand mortar, the material should be rejected. To avoid the removal of any coating on the grains,

* A convenient coefficient of density is the ratio of the sum of the volumes of materials contained in a unit volume to the total unit volume.

† A natural sand obtained at Ottawa, Ill., passing a screen having 20 meshes and retained on a screen having 30 meshes per lin. in.; prepared and furnished by the Ottawa Silica Company, for 2 cents per lb., f. o. b. cars, Ottawa, Ill., under the direction of the Special Committee on Uniform Tests of Cement of the American Society of Civil Engineers.

which may affect the strength, bank sands should not be dried before being made into mortar, but should contain natural moisture. The percentage of moisture may be determined on a separate sample for correcting weight. From 10 to 40% more water may be required in mixing bank or artificial sands than for standard Ottawa sand to produce the same consistency.

(*b*) *Coarse Aggregate*.—This should consist of crushed stone or gravel which is retained on a screen having holes ¼ in. in diameter, and graded from the smallest to the largest particles; it should be clean, hard, durable, and free from all deleterious matter. Aggregates containing dust, and soft, flat, or elongated particles, should be excluded from important structures.

The maximum size of the coarse aggregate is governed by the character of the construction.

For reinforced concrete and for small masses of unreinforced concrete, the aggregate must be small enough to produce with the mortar a homogeneous concrete of viscous consistency which will pass readily between and easily surround the reinforcement and fill all parts of the forms.

For concrete in large masses, the size of the coarse aggregate may be increased, as a large aggregate produces a stronger concrete than a fine one, although it should be noted that the danger of separation from the mortar becomes greater as the size of the coarse aggregate increases.

Cinder concrete should not be used for reinforced concrete structures. It may be allowable in mass for very light loads or for fire protection purposes. The cinders used should be composed of hard, clean, vitreous clinker, free from sulphides, unburned coal, or ashes.

3.—Water.

The water used in mixing concrete should be free from oil, acid, alkalies, or organic matter.

4.—Metal Reinforcement.

The Committee recommends, as a suitable material for reinforcement, steel filling the requirements for structural steel reinforcement of the specifications adopted by the American Railway Engineering Association (Appendix, p. 430).

Where little bending or shaping is required, and also for reinforcement for shrinkage and temperature stresses, material filling the requirements of the specifications adopted by the American Railway Engineering Association for high-carbon steel (Appendix, p. 430) may be used, adopting the same unit stress as hereinafter recommended for structural grade material.

For the reinforcement of slabs, small beams, or minor details, or for reinforcement for shrinkage and temperature stresses, wire drawn from bars of the grade of rivet steel may be used, with the unit stresses hereinafter recommended.

The reinforcement should be free from excessive rust, scale, or coatings of any character which would tend to reduce or destroy the bond.

IV. Preparing and Placing Mortar and Concrete.

1.—Proportions.

The materials to be used in concrete should be carefully selected, of uniform quality, and proportioned with a view to securing as nearly as possible a maximum density.

(a) Unit of Measure.—The unit of measure should be the cubic foot. A bag of cement, containing 94 lb. net, should be considered the equivalent of 1 cu. ft.

The measurement of the fine and coarse aggregates should be by loose volume.

(b) Relation of Fine and Coarse Aggregates.—The fine and coarse aggregate should be used in such relative proportions as will insure maximum density. In unimportant work it is sufficient to do this by individual judgment, using correspondingly higher proportions of cement; for important work these proportions should be carefully determined by density experiments, and the sizing of the fine and coarse aggregates should be uniformly maintained or the proportions changed to meet the varying sizes.

(c) Relation of Cement and Aggregates.—For reinforced concrete construction, one part of cement to a total of six parts of fine and coarse aggregates measured separately should generally be used. For columns, richer mixtures are generally preferable, and in massive masonry or rubble concrete, a mixture of 1:9 or even 1:12 may be used.

These proportions should be determined by the strength or the wearing qualities required in the construction at the critical period of its use. Experienced judgment based on individual observation and tests of similar conditions in similar localities is an excellent guide as to the proper proportions for any particular case.

For all important construction, advance tests should be made of concrete, of the materials, proportions, and consistency to be used in the work. These tests should be made under laboratory conditions to obtain uniformity in mixing, proportioning, and storage, and in case the results do not conform to the requirements of the work, aggregates of a better quality should be chosen, or richer proportions used to obtain the desired results.

2.—Mixing.

The ingredients of concrete should be thoroughly mixed, and the mixing should continue until the cement is uniformly distributed and the mass is uniform in color and homogeneous. As the maximum density and greatest strength of a given mixture depend largely on thorough and complete mixing, it is essential that the work of mixing should receive special attention and care.

Inasmuch as it is difficult to determine, by visual inspection, whether the concrete is uniformly mixed, especially where limestone or aggregates having the color of cement are used, it is essential that the mixing should occupy a definite period of time. The minimum time will depend on whether the mixing is done by machine or hand.

(*a*) *Measuring Ingredients.*—Methods of measurement of the proportions of the various ingredients should be used which will secure separate and uniform measurements of cement, fine aggregate, coarse aggregate, and water, at all times.

(*b*) *Machine Mixing.*—When the conditions will permit, a machine mixer of a type which insures the uniform proportioning of the materials throughout the mass should be used, as a more uniform consistency can be thus obtained. The mixing should continue for a minimum time of at least 1 min. after all the ingredients are assembled in the mixer.

(*c*) *Hand Mixing.*—When it is necessary to mix by hand, the mixing should be on a water-tight platform, and especial precautions should be taken to turn all the ingredients together at least six times and until they are homogeneous in appearance and color.

(*d*) *Consistency.*—The materials should be mixed wet enough to produce a concrete of such a consistency as will flow into the forms and about the metal reinforcement when used, and which, at the same time, can be conveyed from the mixer to the forms without separation of the coarse aggregate from the mortar.

(*e*) *Retempering.*—Mortar or concrete should not be remixed with water after it has partly set.

3.—Placing Concrete.

(*a*) *Methods.*—Concrete, after the completion of the mixing, should be handled rapidly, and in as small masses as is practicable, from the place of mixing to the place of final deposit, and under no circumstances should concrete be used that has partly set. A slow-setting cement should be used when a long time is likely to occur between mixing and placing.

Concrete should be deposited in such a manner as will permit the most thorough compacting, such as can be obtained by working with a straight shovel or slicing tool kept moving up and down until all

the ingredients have settled in their proper places by gravity, and the surplus water has been forced to the surface. Special care should be exercised to prevent the formation of laitance, which hardens very slowly and forms a poor surface on which to deposit fresh concrete. All laitance should be removed.

Before depositing concrete, the reinforcement should be carefully placed in accordance with the plans, and adequate means provided to hold it in its proper position until the concrete has been deposited and compacted; care should be taken to see that the forms are substantial and thoroughly wetted (except in freezing weather) or oiled, and that the space to be occupied by the concrete is free from débris. When the placing of concrete is suspended, all necessary grooves for joining future work should be made before the concrete has had time to set.

When work is resumed, concrete previously placed should be roughened, thoroughly cleansed of foreign material and laitance, thoroughly wetted, and then slushed with a mortar consisting of one part Portland cement and not more than two parts fine aggregate.

The faces of concrete exposed to premature drying should be kept wet for a period of at least 7 days.

(*b*) *Freezing Weather.*—Concrete should not be mixed or deposited at a freezing temperature, unless special precautions are taken to avoid the use of materials covered with ice crystals or containing frost, and to provide means to prevent the concrete from freezing after being placed in position and until it has thoroughly hardened.

As the coarse aggregate forms the greater portion of the concrete, it is particularly important that this material be heated to well above the freezing point.

(*c*) *Rubble Concrete.*—Where the concrete is to be deposited in massive work, its value may be improved and its cost materially reduced by the use of clean stones thoroughly embedded in the concrete as near together as is possible and still entirely surrounded by concrete.

(*d*) *Under Water.*—In placing concrete under water, it is essential to maintain still water at the place of deposit. The use of tremies, properly designed and operated, is a satisfactory method of placing concrete through water. The concrete should be mixed very wet (more so than is ordinarily permissible) so that it will flow readily through the tremie and into the place with practically a level surface.

The coarse aggregate should be smaller than ordinarily used, and never more than 1 in. in diameter. The use of gravel facilitates mixing and assists the flow of concrete through the tremie. The mouth of the tremie should be buried in the concrete so far that it is at all times entirely sealed and the surrounding water prevented from forcing itself into the tremie; the concrete will then discharge without coming in contact with the water. The tremie should be suspended so that it can be lowered quickly when it is necessary either to choke off or prevent

too rapid flow; the lateral flow should preferably be not more than 15 ft.

The flow should be continuous, in order to produce a monolithic mass and prevent the formation of laitance in the interior.

In large structures it may be necessary to divide the mass of concrete into several small compartments or units, filling one at a time. With proper care, it is possible in this manner to obtain as good results under water as in the air.

V. Forms.

Forms should be substantial and unyielding, so that the concrete shall conform to the designed dimensions and contours; and they should be tight in order to prevent the leakage of mortar.

The time for removal of forms is one of the most important steps in the erection of a structure of concrete or reinforced concrete. Care should be taken to inspect the concrete and ascertain its hardness before removing the forms.

So many conditions affect the hardening of concrete that the proper time for the removal of the forms should be decided by some competent and responsible person, especially where the atmospheric conditions are unfavorable.

It may be stated in a general way that forms should remain in place longer for reinforced concrete than for plain or massive concrete, and that forms for floors, beams, and similar horizontal structures should remain in place much longer than for vertical walls.

When the concrete gives a distinctive ring under the blow of a hammer, it is generally an indication that it has hardened sufficiently to permit the removal of the forms with safety. If, however, the temperature is such that there is any possibility that the concrete is frozen, this test is not a safe reliance, as frozen concrete may appear to be very hard.

VI. Details of Construction.

1.—Joints.

(*a*) *Concrete.*—For concrete construction it is desirable to cast the entire structure at one operation, but as this is not always possible, especially in large structures, it is necessary to stop the work at some convenient point. This point should be selected so that the resulting joint may have the least possible effect on the strength of the structure. It is therefore recommended that the joints in columns be made flush with the lower side of the girders; that the joints in girders be at a point midway between supports, but should a beam intersect a girder at this point, the joint should be offset a distance equal to twice the width of the beam; that the joints in the members of a floor system should in general be made at or near the center of the span.

Joints in columns should be perpendicular to the axis of the column, and in girders, beams, and floor slabs perpendicular to the plane of their surfaces.

Girders should never be constructed over freshly formed columns without permitting a period of at least 2 hours to elapse, thus providing for settlement or shrinkage in the columns.

Shrinkage and contraction joints may be necessary in concrete subject to great fluctuations in temperature. The frequency of these joints will depend, first, on the range of temperature to which the concrete will be subjected, and second, on the quantity and position of the reinforcement. These joints should be determined, and provided for in the design. In massive work, such as retaining walls, abutments, etc., built without reinforcement, contraction joints should be provided at intervals of from 25 to 50 ft. and with reinforcement from 50 to 80 ft. (the smaller the height and thickness, the closer the spacing) throughout the length of the structure. To provide against the structure being thrown out of line by unequal settlement, each section of the wall should be tongued and grooved into the adjoining section. A groove should be formed in the surface of the concrete at vertical joints in walls or abutments.

Shrinkage and contraction joints should be lubricated by either an application of petroleum residuum oil or a similar material, so as to permit a free movement at the joint when the concrete expands or contracts.

The insertion of a sheet of copper or zinc, or even tarred paper, will be found advantageous in securing expansion and contraction at the joint.

(*b*) *Reinforcement.*—Wherever it is necessary to splice tension reinforcement, the length of lap should be determined on the basis of the safe bond stress, the stress in the bar, and the shearing resistance of the concrete at the point of splice; or a connection should be made between the bars of sufficient strength to carry the stress. Splices at points of maximum stress should be avoided. In columns, bars more than $\frac{3}{4}$ in. in diameter, not subject to tension, should be properly squared and butted in a suitable sleeve; smaller bars may be treated as indicated for tension reinforcement, or the stress may be cared for by embedment in large masses of concrete. At foundations, bearing plates should be provided for supporting the bars, or the bars may be carried into the footing a sufficient distance to transmit the stress of the steel to the concrete by means of the bearing and bond resistance; in no case shall the ends of the bars be permitted merely to rest on concrete.

2.—Shrinkage and Temperature Changes.

Shrinkage of concrete, due to hardening and contraction from temperature changes, causes cracks the size of which depends on the

extent of the mass. The resulting stresses are important in monolithic construction, and should be considered carefully by the designer; they cannot be counteracted successfully, but the effects can be minimized.

Large cracks, produced by quick hardening or wide ranges of temperature, can be broken up to some extent into small cracks by placing reinforcement in the concrete; in long continuous lengths of concrete, it is better to provide shrinkage joints at points in the structure where they will do little or no harm. Reinforcement is of assistance, and permits longer distances between shrinkage joints than when no reinforcement is used.

Small masses or thin bodies of concrete should not be joined to larger or thicker masses without providing for shrinkage at such points. Fillets similar to those used in metal castings, but of larger dimensions, for gradually reducing from the thicker to the thinner body, are of advantage.

Shrinkage cracks are likely to occur at points where fresh concrete is joined to that which is set, and hence, in placing the concrete, construction joints should be made on horizontal and vertical lines, and, if possible, at points where joints would naturally occur in dimension-stone masonry.

3.—Fire-Proofing.

The actual fire tests of concrete and reinforced concrete have been limited, but experience, together with the results of tests thus far made, indicates that concrete, on account of its low rate of heat conductivity and the fact that it is incombustible, may be used safely for fire-proofing purposes.

The dehydration of concrete probably begins at about 500° Fahr., and is completed at about 900° Fahr., but experience indicates that the volatilization of the water absorbs heat from the surrounding mass, which, together with the resistance of the air cells, tends to increase the heat resistance of the concrete, so that the process of dehydration is very much retarded. The concrete that is actually affected by fire remains in position and affords protection to the concrete beneath it.

The thickness of the protective coating required depends on the probable duration of a fire which is likely to occur in the structure, and should be based on the rate of heat conductivity. The question of the conductivity of concrete is one which requires further study and investigation before a definite rate for different classes of concrete can be fully established. However, for ordinary conditions, it is recommended that the metal in girders and columns be protected by a minimum of 2 in. of concrete; that the metal in beams be protected by a minimum of 1½ in. of concrete; and that the metal in floor slabs be protected by a minimum of 1 in. of concrete.

It is recommended that, in monolithic concrete columns, the concrete to a depth of $1\frac{1}{2}$ in. be considered as protective covering, and not included in the effective section.

It is recommended that the corners of columns, girders, and beams be beveled or rounded, as a sharp corner is more seriously affected by fire than a round one.

4.—Water-Proofing.

Many expedients have been used to render concrete impervious to water under normal conditions, and also under pressure conditions that exist in reservoirs, dams, and conduits of various kinds. Experience shows, however, that where mortar or concrete is proportioned to obtain the greatest practicable density and is mixed to a rather wet consistency, the resulting mortar or concrete is impervious under moderate pressure.

A concrete of dry consistency is more or less pervious to water, and compounds of various kinds have been mixed with the concrete, or applied as a wash to the surface for the purpose of making it water-tight. Many of these compounds are of but temporary value, and in time lose their power of imparting impermeability to the concrete.

In the case of subways, long retaining walls, and reservoirs, provided the concrete itself is impervious, cracks may be so reduced by horizontal and vertical reinforcement properly proportioned and located, that they are too minute to permit leakage, or are soon closed by infiltration of silt.

Coal-tar preparations, applied either as a mastic or as a coating on felt or cloth fabric, are used for water-proofing, and should be proof against injury by liquids or gases.

For retaining and similar walls in direct contact with the earth, the application of one or two coatings of hot coal-tar pitch to the thoroughly dried surface of concrete is an efficient method of preventing the penetration of moisture from the earth.

5.—Surface Finish.

Concrete is a material of an individual type, and should not be used in imitation of other structural materials. One of the important problems connected with its use is the character of the finish of exposed surfaces. The finish of the surface should be determined before the concrete is placed, and the work should be conducted so as to make possible the finish desired. For many forms of construction the natural surface of the concrete is unobjectionable, but frequently the marks of the boards and the flat dead surface are displeasing, making some special treatment desirable. A treatment of the surface, either by scrubbing it while green or by tooling it after it is hard, which removes the film of mortar and brings the aggregates of the concrete

into relief is frequently used to remove the form markings, break the monotonous appearance of the surface, and make it more pleasing. The plastering of surfaces, as ordinarily applied, should be avoided, for, even if carefully done, it is likely to peel off under the action of frost or temperature changes.

VII. Design.

1.—Massive Concrete.

In the design of massive or plain concrete, no account should be taken of the tensile strength of the material, and sections should usually be proportioned so as to avoid tensile stresses, except in slight amounts to resist indirect stresses. This will generally be accomplished, in the case of rectangular shapes, if the line of pressure is kept within the middle third of the section, but, in very large structures, such as high masonry dams, a more exact analysis may be required. Structures of massive concrete are able to resist unbalanced lateral forces by reason of their weight, hence the element of weight rather than strength often determines the design. A relatively cheap and weak concrete, therefore, will often be suitable for massive concrete structures.

It is desirable, generally, to provide joints at intervals, to localize the effect of contraction.

Massive concrete is suitable for dams, retaining walls, and piers and short columns in which the ratio of length to least width is relatively small. Under ordinary conditions, this ratio should not exceed six. It is also suitable for arches of moderate span, where the conditions as to foundations are favorable.

2.—Reinforced Concrete.

By the use of metal reinforcement to resist the principal tensile stresses, concrete becomes available for general use in a great variety of structures and structural forms. This combination of concrete and metal is particularly advantageous in the beam, where both compression and tension exist; it is also advantageous in the column, where the main stresses are compressive, but where cross-bending may exist. The theory of design, therefore, will relate mainly to the analysis of beams and columns.

3.—General Assumptions.

(*a*) *Loads.*—The loads or forces to be resisted consist of:

1. *The Dead Load.*—This includes the weight of the structure and fixed loads and forces.
2. *The Live Load.*—This consists of the loads and forces which are variable. The dynamic effect of the live load will often

require consideration. Any allowance for the dynamic effect is preferably taken into account by adding the desired amount to the live load or to the live-load stresses. The working stresses hereinafter recommended are intended to apply to the equivalent static stresses thus determined.

In the case of high buildings, the live load on columns may be reduced in accordance with the usual practice.

(*b*) *Lengths of Beams and Columns.*—The span length for beams and slabs shall be taken as the distance from center to center of supports, but need not be taken to exceed the clear span plus the depth of beam or slab. Brackets shall not be considered as reducing the clear span in the sense here intended.

The length of columns shall be taken as the maximum unsupported length.

(*c*) *Internal Stresses.*—As a basis for calculations relating to the strength of structures, the following assumptions are recommended:

1. Calculations will be made with reference to working stresses and safe loads, rather than with reference to ultimate strength and ultimate loads.
2. A plane section before bending remains plane after bending.
3. The modulus of elasticity of concrete in compression, within the usual limits of working stresses, is constant The distribution of compressive stresses in beams, therefore, is rectilinear.
4. In calculating the moment of resistance of beams, the tensile stresses in the concrete are neglected.
5. Perfect adhesion is assumed between concrete and reinforcement. Under compressive stresses, the two materials, therefore, are stressed in proportion to their moduli of elasticity.
6. The ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete is taken at 15, except as modified in Chapter VIII, Section 8.
7. Initial stress in the reinforcement, due to contraction or expansion in the concrete, is neglected.

It is recognized that some of the assumptions given herein are not entirely borne out by experimental data. They are given in the interest of simplicity and uniformity, and variations from exact conditions are taken into account in the selection of formulas and working stresses.

The deflection of beams is affected by the tensile strength developed throughout the length of the beam. For calculations of deflections, a value of 8 for the ratio of the moduli will give results corresponding approximately with the actual conditions.

4.—T-Beams.

In beam and slab construction, an effective bond should be provided at the junction of the beam and slab. When the principal slab reinforcement is parallel to the beam, transverse reinforcement should be used, extending over the beam and well into the slab.

Where adequate bond and shearing resistance between slab and web of beam is provided, the slab may be considered as an integral part of the beam, but its effective width shall be determined by the following rules:

(*a*) It shall not exceed one-fourth of the span length of the beam;

(*b*) Its overhanging width on either side of the web, shall not exceed four times the thickness of the slab.

In the design of T-beams acting as continuous beams, due consideration should be given to the compressive stresses at the support. Beams in which the tee form is used only for the purpose of providing additional compression area of concrete should preferably have a width of flange not more than three times the width of the stem and a thickness of flange not less than one-third of the depth of the beam. Both in this form and in the beam and slab form, the web stresses and the limitations in placing and spacing the longitudinal reinforcement will probably be controlling factors in design.

5.—Floor Slabs.

Floor slabs should be designed and reinforced as continuous over the supports. If the length of the slab exceeds one and five-tenth times its width, the entire load should be carried by transverse reinforcement. Square slabs may well be reinforced in both directions.*

The continuous flat slab with multiple-way reinforcement is a type of construction used quite extensively, and has recognized ad-

* The exact distribution of load on square and rectangular slabs, supported on four sides and reinforced in both directions, cannot readily be determined. The following method of calculation is recognized as faulty, but it is offered as a tentative method which will give results on the safe side. The distribution of load is first to be determined by the formula:

$$r = \frac{l^4}{l^4 + b^4}$$

in which r = proportion of load carried by the transverse reinforcement, l = length, and b = breadth of slab. For various ratios of $\frac{l}{b}$ the values of r are as follows:

$\frac{l}{b}$	r
1	0.50
1.1	0.59
1.2	0.67
1.3	0.75
1.4	0.80
1.5	0.83

Using the values above specified, each set of reinforcement is to be calculated in the same manner as slabs having supports on two sides only, but the total amount of reinforcement thus determined may be reduced 25%, by gradually increasing the rod spacing from the third point to the edge of the slab.

vantages for special conditions, as in the case of warehouses with large, open, floor space. At present, a considerable difference of opinion exists among engineers as to the formulas and constants which should be used, but experience and tests are accumulating data which it is hoped will in the near future permit the formulation of the principles of design for this form of construction.

The loads carried to beams by slabs which are reinforced in two directions will not be uniformly distributed to the supporting beam, and its distribution will depend on the relative stiffness of the slab and the supporting beam. The distribution under ordinary conditions of construction may be expected to be that in which the load on the beam varies in accordance with the ordinates of a parabola having its vertex at the middle of the span. For any given design, the probable distribution should be ascertained, and the moments in the beam calculated accordingly.

6.—Continuous Beams and Slabs.

When the beam or slab is continuous over its supports, reinforcement should be fully provided at points of negative moment, and the stresses in concrete recommended in Chapter VIII, Section 4, should not be exceeded. In computing the positive and negative moments in beams and slabs continuous over several supports, due to uniformly distributed loads, the following rules are recommended:

(*a*) That for floor slabs, the bending moments at center and at support be taken at $\frac{wl^2}{12}$ for both dead and live loads, where w represents the load per linear foot and l the span length.

(*b*) That for beams, the bending moment at center and at support for interior spans be taken at $\frac{wl^2}{12}$, and for end spans it be taken at $\frac{wl^2}{10}$ for center and adjoining support, for both dead and live loads.

(*c*) In the case of beams and slabs continuous for two spans only, the bending moment at the central support should be taken as $\frac{wl^2}{8}$ and near the middle of the span as $\frac{wl^2}{10}$.

(*d*) At the ends of continuous beams, the amount of negative moment which will be developed will depend on the condition of restraint or fixedness, and this will depend on the form of construction used. There will usually be some restraint, and there is likely to be considerable. Provision should be made for the negative bending moment, but, as its amount will depend on the form of construction, the coefficient cannot be specified here, and must be left to the judgment of the designer.

For spans of unusual length, more exact calculations should be made. Special consideration is also required in the case of concentrated loads.

Even if the center of the span is designed for a greater bending moment than is called for by *a* or *b*, the negative moment at the support should not be taken as less than the values there given.

Where beams are reinforced on the compression side, the steel may be assumed to carry its proportion of stress, in accordance with the provisions of Chapter VII, Section 3, *c*-6. In the case of cantilever and continuous beams, tensile and compressive reinforcement over supports must extend sufficiently beyond the support and beyond the point of inflection to develop the requisite bond strength.

7.—Bond Strength, and Spacing of Reinforcement.

Adequate bond strength should be provided. The formula hereinafter given for bond stresses in beams is for straight longitudinal bars. In beams in which a portion of the reinforcement is bent up near the end, the bond stress at places in both the straight bars and the bent bars will be considerably greater than for all the bars straight, and the stress at some point may be several times as much as that found by considering the stress to be uniformly distributed along the bar. In restrained and cantilever beams, full tensile stress exists in the reinforcing bars at the point of support, and the bars must be anchored in the support sufficiently to develop this stress.

In case of anchorage of bars, an additional length of bar must be provided beyond that found on the assumption of uniform bond stress, for the reason that, before the bond resistance at the end of the bar can be developed, the bar may have begun to slip at another point, and "running" resistance is less than the resistance before slip begins.

Where high bond resistance is required, the deformed bar is a suitable means of supplying the necessary strength; but it should be recognized that, even with a deformed bar, initial slip occurs at early loads, and that the ultimate loads obtained in the usual tests for bond resistance may be misleading. Adequate bond strength throughout the length of a bar is preferable to end anchorage, but, as an additional safeguard, such anchorage may properly be used in special cases. Anchorage furnished by short bends at a right angle is less effective than hooks consisting of turns through 180 degrees.

The lateral spacing of parallel bars should not be less than three diameters, from center to center, nor should the distance from the side of the beam to the center of the nearest bar be less than two diameters. The clear spacing between two layers of bars should not be less than 1 in. The use of more than two layers is to be discouraged, unless the layers are tied together by adequate metal connections, particularly at and near points where bars are bent up or bent down.

8.—Diagonal Tension and Shear.

When a reinforced concrete beam is subjected to flexural action, diagonal tensile stresses are set up. If, in a beam not having web reinforcement, these stresses exceed the tensile strength of the concrete, failure of the beam will ensue. When web reinforcement, made up of stirrups, or of diagonal bars secured to the longitudinal reinforcement, or of longitudinal reinforcing bars bent up at several points, is used, new conditions prevail; but even in this case, at the beginning of loading, the diagonal tension developed is taken principally by the concrete, the deformations which are developed in the concrete permitting but little stress to be taken by the web reinforcement. When the resistance of the concrete to the diagonal tension is overcome at any point in the depth of the beam, greater stress is at once set up in the web reinforcement.

For homogeneous beams, the analytical treatment of diagonal tension is not very complex—the diagonal tensile stress is a function of the horizontal and vertical shearing stresses and of the horizontal tensile stress at the point considered, and as the intensity of these three stresses varies from the neutral axis to the remotest fiber, the intensity of the diagonal tension will be different at different points in the section, and will change with different proportionate dimensions of length to depth of beam. For the composite structure of reinforced concrete beams, an analysis of the web stresses, and particularly of the diagonal tensile stresses is very complex; and when the variations due to a change from no horizontal tensile stress in the concrete at the remotest fiber to the presence of horizontal tensile stress at some point below the neutral axis are considered, the problem becomes more complex and indefinite. Under these circumstances, in designing, recourse is had to the use of the calculated vertical shearing stress as a means of comparing or measuring the diagonal tensile stresses developed, it being understood that the vertical shearing stress is not the numerical equivalent of the diagonal tensile stress, and even that there is not a constant ratio between them. It is here recommended that the maximum vertical shearing stress in a section be used as the means of comparison of the resistance to diagonal tensile stress developed in the concrete in beams not having web reinforcement.

Even after the concrete has reached its limit of resistance to diagonal tension, if the beam has web reinforcement, conditions of beam action will continue to prevail, at least through the compression area, and the web reinforcement will be called on to resist only a part of the web stresses. From experiments with beams, it is concluded that it is safe practice to use only two-thirds of the external vertical shear in making calculations of the stresses that come on stirrups, diagonal web pieces, and bent-up bars, and it is here recommended for calculations in de-

signing that two-thirds of the external vertical shear be taken as producing stresses in web reinforcement.

Experiments bearing on the design of details of web reinforcement are not yet complete enough to allow more than general and tentative recommendations to be made. It is well established, however, that vertical members attached to or looped about horizontal members, inclined members secured to horizontal members in such a way as to insure against slip, and the bending of a part of the longitudinal reinforcement at an angle, will increase the strength of a beam against failure by diagonal tension, and that a well-designed and well-distributed web reinforcement may, under the best conditions, increase the total vertical shear carried to a value as much as three times that obtained when the bars are all horizontal and no web reinforcement is used. Where vertical stirrups are used without being secured to the longitudinal reinforcement, the force transmitted between longitudinal bar and stirrup must not be greater than can be taken through the concrete, and care must be taken to provide for the larger bond stress developed in the longitudinal bars with this construction than exists in the absence of stirrups. Sufficient bond resistance between the concrete and the stirrups or diagonals must be provided. Where the longitudinal bars are bent up, the points of bending of the several bars should be distributed along a portion of the length of the beam in such a way as to give efficient web reinforcement over the portion of the length of the beam in which it is needed. The higher resistance to diagonal tension failures given by unit frames having the stirrups and bent-up bars securely connected together both longitudinally and laterally is worthy of recognition. It is necessary that a limit be placed on the amount of shear which may be allowed in a beam; for when web reinforcement sufficiently efficient to give very high web resistance is used, at the higher stresses the concrete in the beam becomes checked and cracked in such a way as to endanger its durability as well as its strength.

The section to be taken as the critical section in the calculation of shearing stresses will generally be the one having the maximum vertical shear, though experiments show that the section at which diagonal tension failures occur is not just at a support, even though the shear at the latter point be much greater.

The longitudinal spacing of stirrups or diagonal members, or the distribution of the points of bending of adjacent bent-up bars, should not exceed three-fourths the depth of the beam.

It is important that adequate bond strength or anchorage be provided to develop fully the assumed strength of all web reinforcement.

It should be noted that it is on the tension side of a beam that diagonal tension develops in a critical way, and that the proper connection must always be made between stirrups or other web reinforcement and the

longitudinal tension reinforcement, whether the latter is on the lower side of the beam or on its upper side. Where negative moment exists, as is the case near the supports in a continuous beam, web reinforcement, to be effective, must be looped over, or wrapped around, or be connected with, the longitudinal tension reinforcing bars at the top of the beam, in the same way as is necessary at the bottom of the beam at sections where the bending moment is positive and the tension reinforcing bars are at the bottom of the beam.

Inasmuch as the smaller the longitudinal deformations in the horizontal reinforcement are, the less the tendency for the formation of diagonal cracks, a beam will be strengthened against diagonal tension failure by arranging and proportioning the horizontal reinforcement so that the unit stresses at points of large shear shall be relatively low.

Where pure shearing stress occurs, or shearing stress combined with but a small amount of tensile stress in the concrete, as when a concentrated load rests on a slab, or other forms of punching shear are produced, or in the case of compression pieces, the element of tension will not need consideration, and the permissible limit of the shearing stress will be higher than the allowable limit when this stress is used as a means of comparing diagonal tensile stress. The working values recommended are given in Chapter VIII, Working Stresses.

9.—Columns.

By columns are meant compression members of which the ratio of unsupported length to least width exceeds about six, and which are provided with reinforcement of one of the forms hereafter described.

It is recommended that the ratio of unsupported length of column to its least width be limited to 15.

The effective area of the column shall be taken as the area within the protective covering, as defined in Chapter VI, Section 3; or, in the case of hooped columns or columns reinforced with structural shapes, it shall be taken as the area within the hooping or structural shapes.

Columns may be reinforced by longitudinal bars, by bands, hoops, or spirals, together with longitudinal bars, or by structural forms which in themselves are sufficiently rigid to act as columns. The general effect of closely spaced hooping is greatly to increase the "toughness" of the column and its ultimate strength, but hooping has little effect on its behavior within the limit of elasticity. It thus renders the concrete a safer and more reliable material, and should permit the use of a somewhat higher working stress. The beneficial effects of "toughening" are adequately provided by a moderate amount of hooping, a larger amount serving mainly to increase the ultimate strength and the possible deformation before ultimate failure.

Composite columns of structural steel and concrete in which the steel forms a column by itself, should be designed with caution.

To classify this type as a concrete column reinforced with structural steel is hardly permissible, as the steel will generally take the greater part of the load. When this type of column is used, the concrete should not be relied on to tie the steel units together or to transmit stresses from one unit to another. The units should be adequately tied together by tie-plates or lattice bars, which, together with other details, such as splices, etc., should be designed in conformity with standard practice for structural steel. The concrete may exert a beneficial effect in restraining the steel from lateral deflection, and also in increasing the carrying capacity of the column. The proportion of load to be carried by the concrete will depend on the form of the column and the method of construction. Generally, for high percentages of steel, the concrete will develop relatively low unit stresses, and caution should be used in placing dependence on the concrete.

The following recommendations are made for the relative working stresses in the concrete for the several types of columns:

(*a*) Columns with longitudinal reinforcement only, to the extent of not less than 1% and not more than 4%: the unit stress recommended for axial compression in Chapter VIII, Section 3.

(*b*) Columns with reinforcement of bands, hoops, or spirals, as hereinafter specified: stresses 20% higher than given for *a*, provided the ratio of the unsupported length of the column to the diameter of the hooped core is not more than 8.

(*c*) Columns reinforced with not less than 1% and not more than 4% of longitudinal bars, and with bands, hoops, or spirals, as hereinafter specified: stresses 45% higher than given for *a*, provided the ratio of the unsupported length of the column to the diameter of the hooped core is not more than 8.

The foregoing recommendations are based on the following conditions:

In all cases, longitudinal reinforcement is assumed to carry its proportion of stress, in accordance with Section 3. The hoops or bands are not to be counted on directly as adding to the strength of the column.

Bars composing longitudinal reinforcement shall be straight, and shall have sufficient lateral support to be securely held in place until the concrete has set.

Where hooping is used, the total amount of such reinforcement shall be not less than 1% of the volume of the column enclosed. The clear spacing of such hooping shall be not greater than one-sixth of the

diameter of the enclosed column, and preferably not greater than one-tenth, and in no case more than 2½ in. Hooping is to be circular, and the ends of bands must be united in such a way as to develop their full strength. Adequate means must be provided to hold bands or hoops in place so as to form a column, the core of which shall be straight and well centered. The strength of hooped columns depends very much on the ratio of length to diameter of hooped core, and the strength due to hooping decreases rapidly as this ratio increases beyond five. The working stresses recommended are for hooped columns with a length of not more than eight diameters of the hooped core.

Bending stresses due to eccentric loads and lateral forces must be provided for by increasing the section until the maximum stress does not exceed the values above specified; and, where tension is possible in the longitudinal bars, adequate connection between the ends of the bars must be provided to take this tension.

10.—Reinforcing for Shrinkage and Temperature Stresses.

When areas of concrete too large to expand and contract freely as a whole are exposed to atmospheric conditions, the changes of form due to shrinkage (resulting from hardening) and to action of temperature are such that cracks may occur in the mass, unless precautions are taken to distribute the stresses so as to prevent the cracks altogether, or to render them very small. The distance apart of the cracks, and consequently their size, will be directly proportional to the diameter of the reinforcement and to the tensile strength of the concrete, and inversely proportional to the percentage of reinforcement and also to its bond resistance per unit of surface area. To be most effective, therefore, reinforcement (in amount generally not less than one-third of 1%) of a form which will develop a high bond resistance should be placed near the exposed surface and be well distributed. The allowable size and spacing of cracks depends on various considerations, such as the necessity for water-tightness, the importance of appearance of the surface, and the atmospheric changes.

VIII. Working Stresses.

1.—General Assumptions.

The following working stresses are recommended for static loads. Proper allowances for vibration and impact are to be added to live loads where necessary to produce an equivalent static load before applying the unit stresses in proportioning parts.

In selecting the permissible working stress to be allowed on concrete, we should be guided by the working stresses usually allowed for other materials of construction, so that all structures of the same class but composed of different materials may have approximately the same degree of safety.

The following recommendations as to allowable stresses are given in the form of percentages of the ultimate strength of the particular concrete which is to be used; this ultimate strength is to be that developed in cylinders 8 in. in diameter and 16 in. long of the consistency described in Chapter IV, Section 2 (*a*), made and stored under laboratory conditions, at an age of 28 days. In the absence of definite knowledge, in advance of construction, as to just what strength may be expected, the Committee submits the following values as those which should be obtained with materials and workmanship in accordance with the recommendations of this report.

Although occasional tests may show higher results than those here given, the Committee recommends that these values should be the maximum used in design.

TABLE OF STRENGTHS OF DIFFERENT MIXTURES OF CONCRETE.

(In Pounds per Square Inch.)

Aggregate.	1:1:2	1:1½:3	1:2:4	1:2½:5	1:3:6
Granite, trap rock	3 300	2 800	2 200	1 800	1 400
Gravel, hard limestone and hard sandstone.	3 000	2 500	2 000	1 600	1 300
Soft limestone and sandstone	2 200	1 800	1 500	1 200	1 000
Cinders.	800	700	600	500	400

NOTE.—For variations in the moduli of elasticity see Chapter VIII, Section 8.

2.—Bearing.

When compression is applied to a surface of concrete of at least twice the loaded area, a stress of 32.5% of the compressive strength may be allowed.

3.—Axial Compression.

For concentric compression on a plain concrete column or pier, the length of which does not exceed 12 diameters, 22.5% of the compressive strength may be allowed.

For other forms of columns the stresses obtained from the ratios given in Chapter VII, Section 9, may govern.

4.—Compression in Extreme Fiber.

The extreme fiber stress of a beam, calculated on the assumption of a constant modulus of elasticity for concrete under working stresses, may be allowed to reach 32.5% of the compressive strength. Adjacent to the support of continuous beams stresses 15% higher may be used.

5.—Shear and Diagonal Tension.

In calculations on beams in which the maximum shearing stress in a section is used as the means of measuring the resistance to diagonal tension stress, the following allowable values for the maximum vertical shearing stress are recommended:

(*a*) For beams with horizontal bars only and without web reinforcement calculated by the method given in the Appendix, Formula (22): 2% of the compressive strength.

(*b*) For beams thoroughly reinforced with web reinforcement: the value of the shearing stress calculated as for *a* (that is, using the total external vertical shear in Formula (22) for shearing unit stress), must not exceed 6% of the compressive strength. The web reinforcement, exclusive of bent-up bars, in this case, shall be proportioned to resist two-thirds of the external vertical shear in the formulas given in the Appendix, Formulas (24) or (25).

(*c*) For beams in which part of the longitudinal reinforcement is used in the form of bent-up bars distributed over a portion of the beam in a way covering the requirements for this type of web reinforcement: the limit of maximum vertical shearing stress (the stress calculated as for *a*), 3% of the compressive strength.

(*d*) Where punching shear occurs, that is, shearing stress uncombined with compression normal to the shearing surface, and with all tension normal to the shearing plane provided for by reinforcement: a shearing stress of 6% of the compressive strength may be allowed.

6.—Bond.

The bond stress between concrete and plain reinforcing bars may be assumed at 4% of the compressive strength, or 2% in the case of drawn wire.

7.—Reinforcement.

The tensile or compressive strength in steel should not exceed 16 000 lb. per sq. in.

In structural steel members, the working stresses adopted by the American Railway Engineering Association are recommended.

8.—Modulus of Elasticity.

The value of the modulus of elasticity of concrete has a wide range, depending on the materials used, the age, the range of stresses between which it is considered, as well as other conditions. It is recommended that in computations for the position of the neutral axis and for the resisting moment of beams and for the compression of concrete in columns it be assumed as:

(*a*) One-fifteenth of that of steel, when the strength of the concrete is taken as 2 200 lb. per. sq. in. or less.

(*b*) One-twelfth of that of steel, when the strength of the concrete is taken as greater than 2 200 lb. per sq. in., or less than 2 900 lb. per sq. in., and

(*c*) One-tenth of that of steel, when the strength of the concrete is taken as greater than 2 900 lb. per sq. in.

Although not rigorously accurate, these assumptions will give safe results. For the deflection of beams which are free to move longitudinally at the supports, in using formulas for deflection which do not take into account the tensile strength developed in the concrete, a modulus one-eighth of that of steel is recommended.

Respectfully submitted,

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DECEMBER, 1912.

IX. Appendix.

1.—Standard Specifications.

(a) *Cement.**

GENERAL OBSERVATIONS.

1.—These remarks have been prepared with a view of pointing out the pertinent features of the various requirements and the precautions to be observed in the interpretation of the results of the tests.

2.—The Committee would suggest that the acceptance or rejection under these specifications be based on tests made by an experienced person having the proper means for making the tests.

SPECIFIC GRAVITY.

3.—Specific gravity is useful in detecting adulteration. The results of tests of specific gravity are not necessarily conclusive as an indication of the quality of a cement, but when in combination with the results of other tests may afford valuable indications.

FINENESS.

4.—The sieves should be kept thoroughly dry.

TIME OF SETTING.

5.—Great care should be exercised to maintain the test pieces under as uniform conditions as possible. A sudden change or wide range of temperature in the room in which the tests are made, a very dry or humid atmosphere, and other irregularities, vitally affect the rate of setting.

CONSTANCY OF VOLUME.

6.—The tests for constancy of volume are divided into two classes, the first normal, the second accelerated. The latter should be regarded as a precautionary test only, and not infallible. So many conditions enter into the making and interpreting of it that it should be used with extreme care.

7.—In making the pats, the greatest care should be exercised to avoid initial strains due to moulding or to too rapid drying-out during the first 24 hours. The pats should be preserved under the most uniform conditions possible, and rapid changes of temperature should be avoided.

8.—The failure to meet the requirements of the accelerated tests need not be sufficient cause for rejection. The cement, however, may be held for 28 days, and a retest made at the end of that period, using a new sample. Failure to meet the requirements at this time should be considered sufficient cause for rejection, although in the present state of our

* Adopted August 16th, 1909, by the American Society for Testing Materials.

knowledge it cannot be said that such failure necessarily indicates unsoundness, nor can the cement be considered entirely satisfactory simply because it passes the tests.

General Conditions.

1. All cement shall be inspected.

2. Cement may be inspected either at the place of manufacture or on the work.

3. In order to allow ample time for inspecting and testing, the cement should be stored in a suitable weather-tight building having the floor properly blocked or raised from the ground.

4. The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment.

5. Every facility shall be provided by the contractor, and a period of at least 12 days allowed for the inspection and necessary tests.

6. Cement shall be delivered in suitable packages, with the brand and name of manufacturer plainly marked thereon.

7. A bag of cement shall contain 94 lb. of cement net. Each barrel of Portland cement shall contain 4 bags, and each barrel of natural cement shall contain 3 bags of the above net weight.

8. Cement failing to meet the 7-day requirements may be held awaiting the results of the 28-day tests before rejection.

9. All tests shall be made in accordance with the methods proposed by the Special Committee on Uniform Tests of Cement of the American Society of Civil Engineers, presented to the Society on January 17th, 1912, with all subsequent amendments thereto.

10. The acceptance or rejection shall be based on the following requirements:

Natural Cement.

11. *Definition.*—This term shall be applied to the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas.

FINENESS.

12. It shall leave by weight a residue of not more than 10% on the No. 100, and 30% on the No. 200 sieve.

TIME OF SETTING.

13. It shall not develop initial set in less than 10 min., and shall not develop hard set in less than 30 min., or more than 3 hours.

TENSILE STRENGTH.

14. The minimum requirements for tensile strength for briquettes 1 sq. in. in cross-section shall be as follows, and the cement shall show no retrogression in strength within the periods specified:

Age.	*Neat Cement.*	Strength.
24 hours in moist air	....	75 lb.
7 days (1 day in moist air, 6 days in water)	....	150 "
28 days (1 day in moist air, 27 days in water)	....	250 "
	One Part Cement, Three Parts Standard Ottawa Sand.	
7 days (1 day in moist air, 6 days in water)	....	50 lb.
28 days (1 day in moist air, 27 days in water)	....	125 "

CONSTANCY OF VOLUME.

15. Pats of neat cement about 3 in. in diameter, ½ in. thick at the center, tapering to a thin edge, shall be kept in moist air for a period of 24 hours.

(*a*) A pat is then kept in air at normal temperature.

(*b*) Another is kept in water maintained as near 70° Fahr. as practicable.

16. These pats are observed at intervals for at least 28 days, and, to pass the tests satisfactorily, should remain firm and hard and show no signs of distortion, checking, cracking, or disintegrating.

PORTLAND CEMENT.

17. *Definition.*—This term is applied to the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials, and to which no addition greater than 3% has been made subsequent to calcination.

SPECIFIC GRAVITY.

18. The specific gravity of cement shall be not less than 3.10. Should the test of cement as received fall below this requirement, a second test may be made on a sample ignited at a low red heat. The loss in weight of the ignited cement shall not exceed 4 per cent.

FINENESS.

19. It shall leave by weight a residue of not more than 8% on the No. 100, and not more than 25% on the No. 200 sieve.

TIME OF SETTING.

20. It shall not develop initial set in less than 30 min.; and must develop hard set in not less than 1 hour, nor more than 10 hours.

TENSILE STRENGTH.

21. The minimum requirements for tensile strength for briquettes 1 sq. in. in cross-section shall be as follows, and the cement shall show no retrogression in strength within the periods specified:

Age.	*Neat Cement.*	Strength.
24	hours in moist air	175 lb.
7	days (1 day in moist air, 6 days in water)	500 "
28	days (1 day in moist air, 27 days in water)	600 "

One Part Cement, Three Parts Standard Ottawa Sand.

7	days (1 day in moist air, 6 days in water)	200 lb.
28	days (1 day in moist air, 27 days in water)	275 "

CONSTANCY OF VOLUME.

22. Pats of neat cement about 3 in. in diameter, ½ in. thick at the center, and tapering to a thin edge, shall be kept in moist air for a period of 24 hours.

(*a*) A pat is then kept in air at normal temperature and observed at intervals for at least 28 days.

(*b*) Another pat is kept in water maintained as near 70° Fahr. as practicable, and observed at intervals for at least 28 days.

(*c*) A third pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel for 5 hours.

23. These pats, to pass the requirements satisfactorily, shall remain firm and hard, and show no signs of distortion, checking, cracking, or disintegrating.

SULPHURIC ACID AND MAGNESIA.

24. The cement shall not contain more than 1.75% of anhydrous sulphuric acid (SO_3), nor more than 4% of magnesia (MgO).

(*b*) *Metal Reinforcement.**

6.—Steel shall be made by the open-hearth process. Re-rolled material will not be accepted.

7.—Plates and shapes used for reinforcement shall be of structural steel only. Bars and wire may be of structural steel or high-carbon steel.

8.—The chemical and physical properties shall conform to the following limits:

Elements Considered.	Structural Steel.	High-Carbon Steel.
Phosphorus, maximum, Basic	0.04 per cent.	0.04 per cent.
Phosphorus, maximum, Acid	0.06 " "	0.06 " "
Sulphur, maximum	0.05 " "	0.05 " "
Ultimate tensile strength. Pounds, per square inch	Desired. 60 000	Desired. 88 000
Elong. min. percentage in 8 in., Fig. 1	1 500 000† / Ult. tensile strength.	1 000 000 / Ult. tensile strength.
Character of fracture	Silky	Silky or finely granular.
Cold bends without fracture	180° flat ‡	180° $d = 4t$ §

* Adopted March 16th, 1910, by the American Railway Engineering Association.
† See Paragraph 15. ‡ See Paragraphs 16 and 17. § "$d = 4t$" signifies "around a pin having a diameter four times the thickness of the specimen."

9.—The yield point for bars and wire, as indicated by the drop of the beam, shall be not less than 60% of the ultimate tensile strength.

10.—If the ultimate strength varies more than 4 000 lb. for structural steel or 6 000 lb. for high-carbon steel, a retest shall be made on the same gauge, which, to be acceptable, shall be within 5 000 lb. for structural steel, or 8 000 lb. for high-carbon steel, of the desired ultimate.

11.—Chemical determinations of the percentages of carbon, phosphorus, sulphur, and manganese shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the engineer or his inspector. Check analyses shall be made from finished material, if called for, in which case an excess of 25% above the required limits will be allowed.

12.—*Plates, Shapes, and Bars.*—Specimens for tensile and bending tests for plates and shapes shall be made by cutting, from the finished product, coupons which shall have both faces rolled and both edges

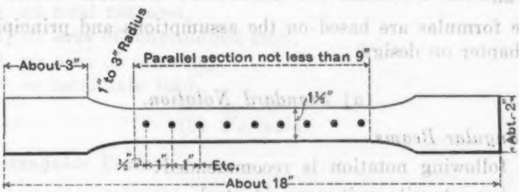

FIG. 1.—TENSION TEST PIECE.

milled to the form shown by Fig. 1; or with both edges parallel; or they may be turned to a diameter of ¾ in. with enlarged ends.

13.—Bars shall be tested in their finished form.

14.—At least one tensile and one bending test shall be made from each melt of steel as rolled. In case steel differing ⅜ in. and more in thickness is rolled from one melt, a test shall be made from the thickest and thinnest material rolled.

15.—For material less than $\frac{5}{16}$ in. and more than ¾ in. in thickness the following modifications will be allowed in the requirements for elongation:

(*a*) For each $\frac{1}{16}$ in. in thickness below $\frac{5}{16}$ in., a deduction of 2½ will be allowed from the specified percentage.

(*b*) For each ⅛ in. in thickness above ¾ in., a deduction of 1 will be allowed from the specified percentage.

16.—Bending tests may be made by pressure or by blows. Shapes and bars less than 1 in. thick shall bend as called for in Paragraph 8.

17.—Test specimens 1 in. thick and greater shall bend cold 180° around a pin, the diameter of which, for structural steel, is twice the

thickness of the specimen, and for high-carbon steel, is six times the thickness of the specimen, without fracture on the outside of the bend.

18.—Finished material shall be free from injurious seams, flaws, cracks, defective edges, or other defects, and have a smooth, uniform, and workmanlike finish.

19.—Every finished piece of steel shall have the melt number and the name of the manufacturer stamped or rolled on it, except that bar steel and other small parts may be bundled, with the above marks on an attached metal tag.

20.—Material which, subsequent to the above tests at the mills, and its acceptance there, develops weak spots, brittleness, cracks, or other imperfections, or is found to have injurious defects, will be rejected, and shall be replaced by the manufacturer at his own cost.

21.—All reinforcing steel shall be free from excessive rust, loose scale, or other coatings of any character which would reduce or destroy the bond.

2.—Suggested Formulas for Reinforced Concrete Construction.

These formulas are based on the assumptions and principles given in the chapter on design.

(a) *Standard Notation.*

1.—Rectangular Beams.

The following notation is recommended:

f_s = tensile unit stress in steel,
f_c = compressive unit stress in concrete,
E_s = modulus of elasticity of steel,
E_c = modulus of elasticity of concrete,
$n = \frac{E_s}{E_c}$,
M = moment of resistance, or bending moment in general,
A = steel area,
b = breadth of beam,
d = depth of beam to center of steel,
k = ratio of depth of neutral axis to effective depth d,
z = depth of resultant compression below top,
j = ratio of lever arm of resisting couple to depth d,
$jd = d - z$ = arm of resisting couple,
p = steel ratio (not percentage).

2.—T-Beams.

b = width of flange,
b' = width of stem,
t = thickness of flange.

3.—Beams Reinforced for Compression.

A' = area of compressive steel,
p' = steel ratio for compressive steel,
f_s' = compressive unit stress in steel,
C = total compressive stress in concrete,
C' = total compressive stress in steel,
d' = depth to center of compressive steel,
z = depth to resultant of C and C'.

4.—Shear and Bond.

V = total shear,
v = shearing unit stress,
u = bond stress per unit area of bar,
o = circumference or perimeter of bar,
Σ_0 = sum of the perimeters of all bars.

5.—Columns.

A = total net area,
A_s = area of longitudinal steel,
A_c = area of concrete,
P = total safe load.

(*b*) *Formulas.*

1.—Rectangular Beams.

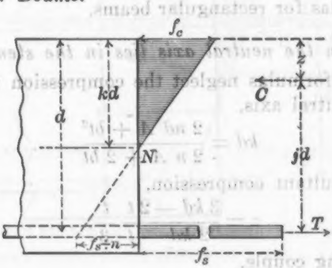

FIG. 2.

Position of neutral axis,

$$k = \sqrt{2\,pn + (pn)^2} - pn \dots\dots\dots\dots\dots\dots (1)$$

Arm of resisting couple,

$$j = 1 - \frac{1}{3}\,k \dots\dots\dots\dots\dots\dots (2)$$

$\left(\text{For } f_s = 15\,000 \text{ to } 16\,000, \text{ and } f_c = 600 \text{ to } 650, k \text{ may be taken at } \frac{3}{8}.\right)$

Fiber stresses,

$$f_s = \frac{M}{Ajd} = \frac{M}{pjbd^2} \quad \text{.......(3)}$$

$$f_c = \frac{2M}{jkbd^2} = \frac{2pf_s}{k} \quad \text{.......(4)}$$

Steel ratio, for balanced reinforcement,

$$p = \frac{1}{2} \frac{1}{\frac{f_s}{f_c}\left(\frac{f_s}{nf_c}+1\right)} \quad \text{.......(5)}$$

2.—T-Beams.

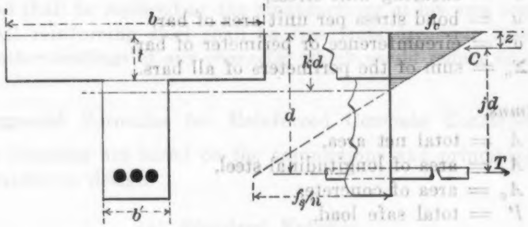

FIG. 3.

Case I. When the neutral axis lies in the flange.
Use the formulas for rectangular beams.

Case II. When the neutral axis lies in the stem.
The following formulas neglect the compression in the stem:
Position of neutral axis,

$$kd = \frac{2\,nd\,A + bt^2}{2\,n\,A + 2\,bt} \quad \text{.......(6)}$$

Position of resultant compression,

$$z = \frac{3\,kd - 2\,t}{2\,kd - t}\,\frac{t}{3} \quad \text{.......(7)}$$

Arm of resisting couple,

$$jd = d - z \quad \text{.......(8)}$$

Fiber stresses,

$$f_s = \frac{M}{Ajd} \quad \text{.......(9)}$$

$$f_c = \frac{Mkd}{bt\left(kd - \frac{1}{2}\,t\right)jd} = \frac{f_s}{n}\,\frac{k}{1-k} \quad \text{.......(10)}$$

(For approximate results, the formulas for rectangular beams may be used.)

The following formulas take into account the compression in the stem; they are recommended where the flange is small compared with the stem:

Position of neutral axis,

$$kd = \sqrt{\frac{2\,nd\,A + (b - b')\,t^2}{b'} + \left(\frac{nA + (b - b')\,t}{b'}\right)^2} - \frac{nA + (b - b')\,t}{b'} \quad \ldots\ldots (11)$$

Position of resultant compression,

$$z = \frac{\left(kdt^2 - \frac{2}{3}\,t^3\right) b + \left[(kd - t)^2 \left(t + \frac{1}{3}\,(kd - t)\right)\right] b'}{t\,(2\,kd - t)\,b + (kd - t)^2\,b'} \ldots (12)$$

Arm of resisting couple, $jd = d - z$ ....(13)

Fiber stresses,

$$f_s = \frac{M}{Ajd} \ldots\ldots (14)$$

$$f_c = \frac{2\,Mkd}{[(2\,kd - t)\,bt + (kd - t)^2\,b']\,jd} \ldots\ldots (15)$$

3.—Beams Reinforced for Compression.

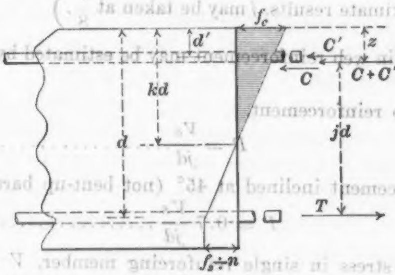

FIG. 4.

Position of neutral axis,

$$k = \sqrt{2\,n\left(p + p'\frac{d'}{d}\right) + n^2\,(p + p')^2} - n\,(p + p') \ldots (16)$$

Position of resultant compression,

$$z = \frac{\frac{1}{3}\,k^3\,d + 2\,p'\,nd'\left(k - \frac{d'}{d}\right)}{k^2 + 2\,p'\,n\left(k - \frac{d'}{d}\right)} \ldots\ldots (17)$$

Arm of resisting couple,

$$jd = d - z \qquad (18)$$

Fiber stresses,

$$f_c = \frac{6\,M}{bd^2\left[3\,k - k^2 + \frac{6\,p'\,n}{k}\left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)\right]} \qquad (19)$$

$$f_s = \frac{M}{pjbd^2} = nf_c\,\frac{1-k}{k} \qquad (20)$$

$$f_s' = nf_c\,\frac{k - \frac{d'}{d}}{k} \qquad (21)$$

4.—Shear, Bond, and Web Reinforcement.

In the following formulas, Σ_0 refers only to the bars constituting the tension reinforcement at the section in question, and jd is the lever arm of the resisting couple at the section.

For rectangular beams,

$$v = \frac{V}{bjd} \qquad (22)$$

$$u = \frac{V}{jd\,\Sigma_0} \qquad (23)$$

(For approximate results, j may be taken at $\frac{7}{8}$.)

The stresses in web reinforcement may be estimated by the following formulas:

Vertical web reinforcement,

$$P = \frac{Vs}{jd} \qquad (24)$$

Web reinforcement inclined at 45° (not bent-up bars),

$$P = 0.7\,\frac{Vs}{jd} \qquad (25)$$

in which P = stress in single reinforcing member, V = amount of total shear assumed as carried by the reinforcement, and s = horizontal spacing of the reinforcing members.

The same formulas apply to beams reinforced for compression as regards shear and bond stress for tensile steel.

For T-beams,

$$v = \frac{V}{b'\,jd} \qquad (26)$$

$$u = \frac{V}{jd\,\Sigma_0} \qquad (27)$$

(For approximate results, j may be taken at $\frac{7}{8}$.)

5.—Columns.

Total safe load,

$$P = f_c (A_c + nA_s) = f_c A (1 + (n - 1) p) \quad \text{(28)}$$

Unit stresses,

$$f_c = \frac{P}{A (1 + (n - 1) p)} \quad \text{(29)}$$

$$f_s = n f_c \quad \text{(30)}$$

AMERICAN SOCIETY OF CIVIL ENGINEERS

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TRANSACTIONS

Paper No. 1290

THE EFFECT OF SATURATION ON THE STRENGTH OF CONCRETE.*

BY J. L. VAN ORNUM, M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. E. A. MORITZ, J. R. WORCESTER, WALTER S. WHEELER, CLIFFORD RICHARDSON, W. K. HATT, HENRY H. QUIMBY, AND J. L. VAN ORNUM.

The paucity of recorded information concerning the treatment of concrete specimens, with regard to moisture conditions during their storage while awaiting the test for strength, seems to indicate a general supposition that this feature has no considerable effect on results. Apparently corroborating this attitude is the statement in the recent report of the Special Committee on Concrete and Reinforced Concrete, in which, while specifying the exact dimensions, mixing, consistency, age, etc., of test specimens, the only requirement designed to control moisture treatment during their curing seems to be that they shall be "stored under laboratory conditions." It is the purpose of this paper to invite attention, not only to the great importance of specifying and standardizing the moisture treatment of specimens intended for testing, but also to the further fact that similar conditions, as they act on the finished structures, will affect their strength considerably, and therefore should be considered in specifying the proper unit stresses. It is evident that this factor should not be ignored when great variations in strength, to an amount of perhaps 50% above or below a mean value, result from differences in moisture conditions.

* Presented at the meeting of November 5th, 1913.

During the last six years this question, at times, has been the subject of investigation in the Washington University Testing Laboratory. Different features have been explored experimentally, as work on graduation theses, since 1907 by Messrs. Trelease, Feinberg, Harris, Start, Bank, Caplan, Bryan, and Keller. Although these tests leave the greater part of the field still untouched, the writer believes the results thus far obtained to be so definite in their showing of a decided influence of moisture conditions on strength, and so significant in their general indications, that he offers this summarized statement of experimental results to the Engineering Profession for its consideration.

The most important part of the investigation is that of the effect, on the compressive strength of concrete, produced by varying systematically the relative length of time of exposure in air and in water.

The test specimens were cylindrical, 8 in. in diameter and 16 in. high. The materials were: a standard brand of Portland cement which fulfilled thoroughly all the requirements of the standard specifications; a clean sand of good quality, weighing about 100 lb. per cu. ft. when dry, and having 36% voids; and a washed river gravel of the same weight, varying in diameter from quite small up to 1½ in., and having 33% voids. The proportions were the usual 1:2:4, by volume; the mixing was done thoroughly by hand; and the quantity of water used was such as to give a moderately wet consistency, which allowed a thorough compacting by stirring with an iron rod and a slight tamping. All details of fabrication, curing, and testing were planned so as to secure such complete uniformity as is practicable to obtain in all regards except the one for which the controlled variation formed the particular purpose of the experimental study.

The cylinders were removed from the moulds when 2 days old, and were tested at an age of 6 weeks. The intervening 40 days constituted the period in which the duration of their immersion in water was varied systematically from nothing to the full time. The average results of the 240 tests thus made are plotted on the diagram, Fig. 1, on which the abscissas represent that number of days (after the 2 days in the moulds and the time of exposure to air) during which each set of specimens was placed in water before crushing them; and the ordinates give the percentage of strength which each set of immersed cylinders (standing in water for the indicated number of days) was found to have, taking the compressive strength of the dry specimens

from the same mix as 100 per cent. Thus, at the extreme left is represented the basis of comparison, or those which were not immersed at all; those specimens which were cured in air of ordinary humidity for 32 days and then immersed for 8 days are shown by the black circle to be 86% as strong as the air-cured concrete; those in air for 12 days

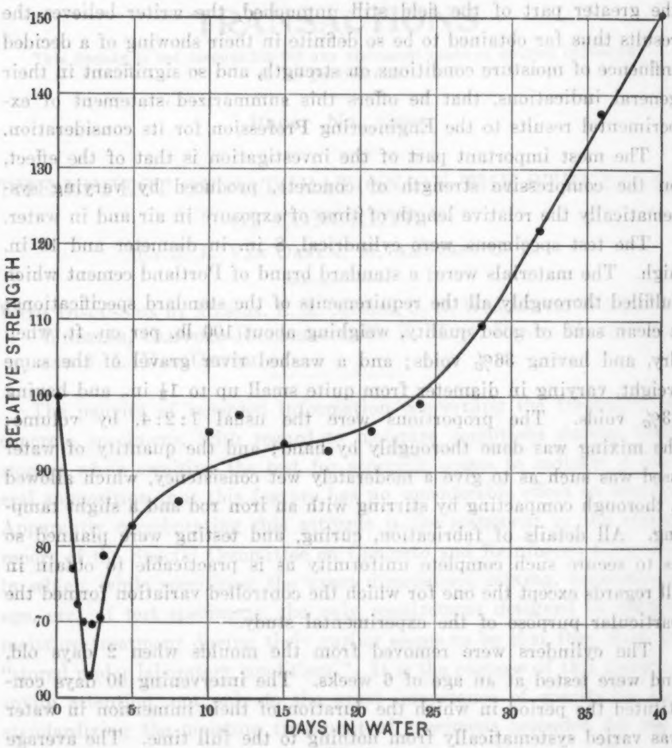

FIG. 1.

and therefore finally cured in water for 28 days have gained 9% in strength; and those submerged for the entire 40 days exhibited an average compressive strength fully 50% greater than that of the air-cured specimens.*

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* Some other experiments have also shown that concrete aged entirely in water is considerably stronger than when left continuously in air, as reported by Rafter, Withey, and the Watertown Arsenal.

An average curve for the plotted points has been drawn as a full line, showing the systematic increase in strength as the time of submergence is lengthened beyond 2 days; but there exists the significant fact that specimens, of the dimensions used, decrease rapidly in strength when stored in air for 38 (or more) days and then placed in water for the remaining 2 days (or less). This particular feature of the rapid loss of strength on first exposure to water, and the active but slower recovery of strength as soaking continued, required a multiplication of tests to determine satisfactorily the locus of the curve in this region; and consequently more than half of the experiments were concentrated in this descending and the adjacent rising portion of the plot.

It thus appears that the compressive strength of concrete exposed only to air may be reduced nearly 40% when saturated with water, but that this loss is actively regained as the treatment is continued. The word saturation is used advisedly, because the minimum strength was found to coincide practically with the length of time required for water to penetrate to the middle of the specimens. Very plainly, this loss of strength has no relation to the percentage of contained moisture, as it is not only regained but much exceeded if the saturation is continued long enough. Perhaps the reduction in strength is purely a temporary physical phenomenon which is gradually counteracted and finally dominated by continued saturation permitting the imperfectly developed chemical processes of hardening to proceed actively. If this be true, concrete would regain something more than its original strength if dried out as soon as completely saturated, but this value would be less than that attainable by a continuance of the water treatment; also a repetition of soaking after such an experience would again temporarily reduce the strength, but less than before. These questions, as well as others, such as the duration of saturation necessary to prevent the temporary relapse of strength described, the corresponding effects of other periods of treatment similar to that discussed, of alternating the exposure to air and water, the result of different dimensions, proportions, materials, etc., all offer a large, interesting, and fruitful field for investigation.

In a series of experiments on the factors affecting the strength of bond between concrete and embedded steel, 74 tests were made to determine whether there existed the same tendency of rapid weakening at first and a following recovery of strength when the dry specimens

were immersed in water. The concrete was of the same character as already described, and the test specimens were partly of the notched beam type transversely loaded, and partly of cylindrical form in which the rod was pulled from the embedding concrete. All care was used to make the methods of testing such as to minimize all variables except the particular one the effect of which was sought. The results indicate clearly, for both plain and deformed bars, that the bond strength values, similarly, decline rapidly and then increase after saturation is complete, as is the case with the compressive strength; although the average minimum observed was only about 75% of that of the specimens cured entirely in air. Whether or not this percentage really marks the greatest weakening of bond produced by immersion at an age of 6 weeks is somewhat uncertain; perhaps additional tests for intermediate periods of soaking would have developed a further reduction in strength. At any rate, a similar behavior characterizes the bond values obtained during the first few days of saturation.

Thirty-two beams were made of such dimensions and amount of longitudinal reinforcement (without any web reinforcement whatever) that failure would always occur through the effect of the excessive web tension in the concrete. The materials were of the same quality as those already described, and equal precautions were taken to secure reliable results. These beams were also tested at an age of 6 weeks, but the small number restricted the investigation to lengths of immersion designed to detect only the early loss of web tensile strength and its subsequent increasing value, without tracing it throughout successively lengthening periods of exposure to water to the limit of 40 days. The characteristic effect is again the same, the lowest average found being again practically three-fourths of the strength of the air-cured specimens. It may be that, in this case also, the minimum value was not detected.

A series of experiments on concrete prisms when 7 years old, to determine any change due to age in elastic properties, has been discussed previously by the writer.* It may be stated, in reference thereto, that the modulus of elasticity of these old prisms exhibited a practically constant value throughout the repeated loadings equal to the maximum before found, which was about 80% greater than the final

* *Transactions*, Am. Soc. C. E., Vol. LVIII, pp. 312-13.

constant value as then reported for prisms of ordinary age; or a value of 4 000 000 in compression for that 1:3:5 limestone concrete. In these experiments two specimens were immersed in water until saturated and then carefully tested; the resulting compressive modulus of elasticity for wet concrete was 60% of that observed on the same specimens when dry. This lowering in value refers, again, only to the effect produced as soon as the saturation is complete; and has no reference to a continuance of the exposure to water, such as is reported on certain other tests,* where the figures given for the compressive modulus of elasticity of concrete specimens cured entirely in water for 26 days are about one-fourth greater than for those cured only in air.

As the various strength values of dry concrete are temporarily reduced from 25 to 40% by saturation, it would seem that this fact should be given definite consideration in fixing the working stresses used in the design of structures which may be thus exposed, or else conditions should be controlled in such a way as to prevent the weakening thus produced. No such effect occurs in concrete constantly under water or in moist earth from the time of its fabrication; but construction above ground, and therefore exposed to dry air for a time and then to a heavy rain or other source of rapid wetting, presents conditions under which this reduction in strength exists temporarily. Fortunately, the remedy is simple and inexpensive. It is to keep the exposed material thoroughly wet until its enclosure by exterior walls and roof renders its saturation by rain impossible. The case of parts not thus protected, or those for which enclosure is delayed, is not so simple; because the length of time of saturation which will make the concrete safe against serious reduction of strength is uncertain. The systematic wetting of concrete is a well-known principle of good construction; but the writer's observation and experience suggest a very considerable tendency to regard that procedure as abstractly correct, but practically rather specious or trivial. One purpose of this paper is to present the facts in such a way that the frequent, thorough, and faithful wetting of all parts of such concrete structures shall henceforth be no more ignored than is now the protection from freezing or disturbance while setting. Probably this treatment should be continued

* *Bulletin No. 175*, University of Wisconsin, p. 17.

for a length of time substantially greater than that heretofore indicated—perhaps for a period expressed in weeks instead of days.

Undoubtedly, carelessness in a thorough control of this kind is a frequent contributing cause of weakness which is sometimes sufficient to result in failure. Very evidently, this temporary weakening of concrete by saturation is amply covered by the factor of safety required by good practice, if it be the only fault; but the materials may be considerably below standard, or the workmanship may be defective, or the design may encroach on the reserve of safety, or the occasional overload may be imposed; and if the material man, the construction superintendent, the designer, and the user of the structure should each rely on the others to meet fully the requirements, in the expectation that his own delinquency will be safely covered by the factor of safety, it would not require an impossible coincidence of such conditions to cause disaster, especially in view of the fact that considerable variations from the average strength values, which form the basis of design, necessarily exist in different parts of the structure. In fact, the failures which have occurred are generally a result of several such contributing causes. The writer believes that the considerable weakening produced by the saturation of dry concrete has invariably been a contributing factor in all those instances in which there was an active wetting of dry or partly dry concrete when subjected to essential stresses.

This general proposition furnishes one more evidence of the remarkable responsiveness of concrete to variations in its treatment. The fact that differences in control (which to the average artisan are seemingly unimportant) actually do exert a positive influence on its essential characteristics, constitutes a definite warning against entrusting it to the uncertainties of irresponsible or skeptical supervision, and assures ample reward for a competent control which is correctly adapted to develop its capabilities. The susceptibility of steel to the influence of phosphorus and sulphur, of details of its heat treatment, and of other conditions occurring in the process of its manufacture, have resulted in restricting its production to the scrutiny of expert superintendence. Equal reason exists for, and commensurate advantages will follow, a thoroughly discriminating control of both the initial fabrication of concrete, and the details of treatment during its hardening, in order to realize the great possibilities inherent in this newer material.

The treatment of steel is not always complete as it comes from the rolls, as is shown by such effects as the changes in strength produced by the cold-twisting of steel rods; much more important in relation to the resulting quality of concrete is the nature of its treatment after fabrication, both because its attainment of strength is a relatively slow process and for the reason that the nature of the prevailing conditions provided during this period affects so greatly the development of its essential properties.

The notable responsiveness of concrete to the character of its treatment is a direct appeal for thoroughly trustworthy and expert control.

DISCUSSION

Mr. Moritz.

E. A. Moritz, Assoc. M. Am. Soc. C. E. (by letter).—This paper recalls some experiments conducted by the writer in 1904 and 1905 at the University of Wisconsin and published in 1906.* It was believed by the writer at that time that concrete stored in water before testing is much stronger than when stored in air for the same period of time, and he was much surprised then, and has been more surprised since, that this point has not been generally recognized in specifications for test pieces.

The writer's experiments were very limited, and were only incidental to some tests of large beams, but the results show up so forcibly the difference in strength of concrete specimens stored in water and those stored in air that it is thought worth while to publish them again in connection with Mr. Van Ornum's paper. The figures in Table 1 are taken from page 364 of the *Bulletin* mentioned. The figures in the last column are the reciprocals of those in the *Bulletin*, and are given in order to make them directly comparable with Mr. Van Ornum's figures.

TABLE 1.

Proportions of concrete.	Unit Stress on Cube:		Ratio, $\frac{S_1}{S_2}$
	In water, S_1.	In air, S_2.	
1 : 2 : 5...........	1 450	1 170	1.23
"	2 720	1 420	1.92
"	2 060	1 600	1.28
"	2 560	1 310	1.96
"	2 560	1 290	2.00
"	2 850	1 620	1.75
"	2 620	1 560	1.70
1 : 3 : 6.5...........	1 950	990	1.96
"	1 620	950	1.70
"	1 180	780	1.51
"	1 520	1 020	1.49
"	1 610	1 050	1.54

All compression tests were made with 4-in. cubes, and at the age of 30 days. The cubes were taken from the moulds at the age of 48 hours, after which one from each batch was stored in air at a temperature of about 65°, and the other stored in water until tested. The concrete was mixed moderately wet—not sloppy. All materials were proportioned by weight and mixed by hand. Natural pit sand and crushed limestone formed the aggregate.

* *Bulletin No. 148*, University of Wisconsin.

J. R. WORCESTER, M. AM. SOC. C. E. (by letter).—The author has brought out in a forceful manner the possible loss of strength in concrete by saturation after premature drying, and, at the same time, the possible gain in strength by continuous saturation up to an age of 6 weeks. This series of tests furnishes ample evidence of the necessity of standardizing the method of curing specimens for compression tests in the laboratory, and may account for an appreciable part of the discrepancies often noticed between the results of experiments made in different laboratories. It also shows that by yielding to the temptation to allow concrete in actual construction to dry out as rapidly as possible, one runs the risk of permanently losing a very material proportion of the strength which might be attained, even though, by its location within a building, there is no danger of the loss of strength through subsequent saturation. Builders are very apt to strip forms from vertical surfaces as quickly as possible, to hasten this drying out, and engineers are likely to overlook the ill effect of this practice. Mr. Worcester.

It seems evident that the treatment to which the test specimens were subjected was calculated to produce unusually rapid drying out, and that the strength of those tested without subsequent immersion, taken as 100 on the scale of relative strengths, must have been rather lower than would naturally be expected of this quality of concrete. If so, the later gain in strength after continuous saturation is easily explained. The author, perhaps, will give us additional light on this point.

The temporary loss of strength on immersion appears to the writer to be analogous to the loss in strength of dried clay when water-soaked. It seems as if some of the cement was dried out so rapidly that it acquired a strength from drying and not from chemical crystallization. The formation of a hardened, light colored skin on the outside of a wall while the interior is still dark, moist, and soft, is a common phenomenon. If this is not the true explanation, the writer hopes that the discussion will bring out a better one.

Although the report of the Special Committee on Concrete and Reinforced Concrete does not elaborate the dangers of too rapid drying, as it might have done to advantage, it does include one recommendation, apparently overlooked by the author, that "The faces of concrete exposed to premature drying should be kept wet for a period of at least 7 days." It would be interesting to know what would have been the effect on the author's experiments of this initial treatment.

In actual construction it seems probable that generally a better chance for hydration exists than was given to these specimens. A cylinder, 8 in. in diameter and 16 in. high, removed from the mould in 2 days and exposed to air of ordinary humidity, would present a

Mr. Worcester. large surface to the effect of evaporation, and would dry out very rapidly. The exposed surface, assuming the cylinders to have stood on end, would have amounted to about 6.75 sq. ft. for each cubic foot of volume. In practice, prior to the removal of the forms, a floor slab would offer the largest exposed surface per unit of volume of any ordinary construction. In this case, with a thickness of 6 in., the exposed area would only be 2 sq. ft. for every cubic foot of volume, and, if the surface were kept wet for 7 days, as it should be, the chance for drying out would be much less than in the test cylinders. It appears, therefore, that though the danger of a decrease in strength from saturation is unlikely to prove serious in practice, it cannot justifiably be ignored.

Mr. Wheeler. WALTER S. WHEELER, M. AM. SOC. C. E. (by letter).—The writer has often noticed that in finished structures new concrete appears to lose part of its strength when first submerged in water. For example, he has just completed some construction in which the proportion was 1 part cement to 3 parts sand, by volume. When 2 or 3 days old, the top of this structure was submerged to a depth of about 1 ft. Before submersion, it was noted that, by pressing with the thumb on the corners and edges of the structure, the concrete could be broken quite easily, and that for several days after submersion it could be broken more easily; in fact, for a few days, it seemed that the structure had only about enough strength to hold together.

After a submersion of 2 months, the structure was as firm as could be expected, and nothing but a charge of dynamite could mar it. The materials used were standard, and the placing was done according to standard practice.

Mr. Richardson. CLIFFORD RICHARDSON, M. AM. SOC. C. E. (by letter).—The conclusions arrived at by Mr. Van Ornum can be confirmed by the experience of the writer, who, however, would go farther and state that a reversal of the conditions with which he has experimented would result in the diminution of the strength of the concrete; that is to say, that a concrete which has been immersed in water for some time loses strength on being dried out. It is to be hoped that Mr. Van Ornum, through some of his students, will conduct some experiments in this as well as in other directions where modification of the environment of concrete may take place. Mr. Van Ornum's paper is valuable, as it brings to the attention of users of concrete a situation which has not been sufficiently appreciated in the past, although it has been known, in a general way, to most engineers.

Mr. Hatt. W. K. HATT, M. AM. SOC. C. E. (by letter).—The writer is interested in and appreciates Professor Van Ornum's experimental inquiry. An intelligent use of materials must be promoted by a knowledge of their underlying properties and essential nature. This knowledge is

more likely to be gained by investigations of the character described in this paper than by elaborate and extensive empirical tests, mechanically planned and performed, which add to the work of the compiler and to the accumulation of débris that is carried along in textbooks and handbooks from one generation to another. Mr. Hatt.

The writer believes that, in this and also in a former paper,* Professor Van Ornum has failed to do justice to his tests, in that he has chosen not to accept the standards imposed by most experimenters upon themselves with respect to completeness of publication of the data. Every one knows that different persons may derive conflicting conclusions from the same data when the full circumstances are known. In the case of this paper, we must accept or reject deductions in respect to an important fundamental matter on the basis of a single diagram. The writer does not desire to be understood as saying that the diagram, with its experimental points and curve, is not the only expression of a law to be obtained from the data which are not published, but he expresses the wish that he, with others, might review the experimental facts independently. Much could also be learned from a report of the measurements of deformations and sets. Only a partial view is obtained from the compressive strength alone.

TABLE 2.—DEFLECTION OF REINFORCED CONCRETE BEAMS UNDER REPEATED LOADINGS.

Deflections and Set, in Inches.

Approximate stress in steel.	Load, in pounds.	Beam No. 12. Number of times applied.	Beam No. 12. Deflection.	Beam No. 12. Set.	Beam No. 15. Times applied.	Beam No. 15. Deflection.	Beam No. 15. Set.	Under constant load for five months. Deflection.	Set.	Ordinary testing machine load.
3 000	2 400	1	0.025	0.005	1	0.025	......	0.130	0.06	0.050
		710	0.035	0.005	...	......	......	0.210		
8 000	5 200	1	0.070	0.010	1	0.070	0.010	0.160	0.13	0.08
		500	0.170	0.060	500	0.140	0.050			
16 000	6 750	1	0.210	0.060	1	0.180	0.060	0.290*	....	0.14
		700	0.260	0.100	450	0.240	0.080			
29 700	8 100	1	0.300	0.100	1	0.270	0.090	0.490	0.36	0.21
		470	0.310	0.120	620	0.290				

* Two months.

In 1906-07 the writer conducted an investigation on the effect of the time element in loading reinforced concrete beams.† Table 2, from that source, shows a constantly increasing deflection under con-

* "The Fatigue of Concrete," *Transactions*, Am. Soc. C. E., Vol. LVIII, p. 294.

† "Notes on the Effect of Time Element in Loading Reinforced Concrete Beams," *Transactions*, Am. Soc. for Testing Materials, Vol. VII, 1907.

Mr. Hatt. tinued load applied to such beams, and also that a repeated application of load produces permanent deflection and set without affecting the subsequent maximum load carried by the beam. In the writer's view, these facts expressed a plasticity in concrete.

In these cases the results could hardly be explained by the effect of water in softening the concrete, as reported by Professor Van Ornum. The beams were exposed outdoors, were protected from rain by a platform, and were tested under repetitive loads in the laboratory.

In arriving at a final view, some help is obtained from the behavior of other materials. The writer has noted an analogy between the behavior of concrete and wood with respect to the effects of certain conditions of loading.

In case of any material, one should know the effect of the following elements on the strength and deformation, both elastic and plastic:

(1). Time-rate of application of stress, including impact loading and long-continued steady loads;

(2). Moisture conditions;

(3). Temperature of material under (1).

(1). Wood exhibits an increasing deformation and set under loads which are continued over periods of days and weeks, and the final load is not affected by precedent sets. Mr. H. D. Tiemann has investigated this matter exhaustively.* Portland cement concrete also possesses this property.

As the speed of application of the load is increased, wood exhibits a higher elastic limit, which, under impact, may be double the static elastic limit. It is evidently affected by the time-rate of the application of loading. No data are known to the writer to determine the effect of speed of loading on concrete.

The final strength of both wood and concrete appears to be unaffected by precedent plastic changes which produce set.

(2). Wood also becomes more plastic when the wood substance contains more water. It is strong and stiff when dry, and weak and flexible when wet. Omitting the difference of the chemical action proceeding in concrete when in water, it appears from Professor Van Ornum's paper that concrete becomes weak when saturated with water. His deformation measurements, not reported, might show whether or not the modulus of elasticity was correspondingly decreased.

After wood is dried out and the excess moisture is removed, it nearly resumes its original properties. Professor Van Ornum suggests that concrete would regain something more than its original strength if dried out after being completely saturated.

* A report of part of his investigations will be found in *Transactions*, Am. Soc. for Testing Materials, Vol. IX (1909), p. 534, "Some Results of Dead Load Bending Tests of Timber by Means of a Recording Deflectometer"; and Vol. VIII (1908), p. 541, "The Effect of the Speed of Testing upon the Strength of Wood and the Standardization of Tests for Speed."

(3). No data are known to the writer to show conclusively the effect of temperature on the strength and deformation of either wood or concrete. Mr. Hatt.

The foregoing discussion indicates that the binding substance in wood and concrete exhibits marked plasticity, not alone under long-continued loadings at low stress, and that this substance is temporarily affected by the absorption of water.

Leaving the field of the writer's knowledge, he would take the liberty of referring to the views of others who consider the hardening of concrete as a phenomenon of colloidal action, and to the statement of those wise in the knowledge of wood substance that a colloidal material binds wood structure together. If this is so, we might expect, from the properties of colloids, that concrete and wood would show the noticed plasticity under long-continued loadings, the noticed effect of moisture on the strength; both should show a difference of behavior under quickly applied loads; and their strength should also be affected by temperature—being more brittle at low temperatures. The writer may be pardoned for sketching in this uncertain field.

Do not the author's results bear out the common view of workers in concrete that some part of the hardening of the concrete floor of a building under construction when exposed to the drying conditions of the atmosphere is due to this "drying out", irrespective of temperature? The writer has noticed the rapid gain in strength of concrete specimens shipped to the laboratory in tin forms after the latter had been removed. Specimens of concrete taken from building failures also gain strength rapidly when exposed to the air of the laboratory. This is no doubt due to hastened chemical action consequent on the increased temperature, but it also may be the process of drying out analogous to that prevailing in wooden beams. Professor Van Ornum coincides in this common view in his statement:

"In fact, the failures which have occurred are generally a result of several such contributing causes. The writer believes that the considerable weakening produced by the saturation of dry concrete has invariably been a contributing factor in all those instances in which there was an active wetting of dry or partly dry concrete when subjected to essential stresses."

Henry H. Quimby, M. Am. Soc. C. E.—Mr. Worcester's discussion brings up recollections of some early experience with concrete. Certain 6-in. test cubes made by an inspector in the field yielded very unsatisfactory results when crushed, breaking as low as 500 lb. per sq. in. The cement, which was a standard Portland brand, and the sand, which was New Jersey bank sand, were suspected and retested, but were found to be normal. The concrete was friable—the edges easily rubbed off with the thumb—and it was very absorbent. Tensile test briquettes of the mortar—1 to 3—made at the same time as the cubes, tested as Mr. Quimby.

Mr. Quimby. low as 39 lb. at 7 days. Some broken pieces partly immersed in water to observe their absorption were noticed after a few days to have considerably higher tenacity. Similar experiences on other work later, and also some experiments, made it clear that premature setting and drying out prevents concrete from properly ripening.

The trouble appears in hot weather, when all the materials have been heated by exposure to the sun, and perhaps when, in addition, a quick-setting cement is used. When concrete is found to be deficient in strength because of premature drying out, continued saturation with water will increase its strength, but will probably not raise it to the normal point unless the saturation be continued for a very long time.

It is evident that concrete wants its moisture and time together.

In order to secure uniform conditions of treatment of test cubes, the rule has been adopted for all inspectors to bury each cube in moist earth as soon as it is removed from the mould, and keep it there until it is sent in to the testing laboratory to be crushed.

Mr. Van Ornum. J. L. Van Ornum, M. Am. Soc. C. E. (by letter).—It is gratifying to note the several instances cited as showing the marked change in strength which results from saturating concrete which has been previously exposed in air. Undoubtedly there are many others, that have not been reported, which would indicate the same general tendency. Of course, the proportionate loss of strength occurring when an air-cured concrete is saturated would depend on a multiplicity of existing conditions, some of which have been referred to. One of these is the relatively large area of surface compared to volume in the test specimens, as noted by Mr. Worcester. It would seem probable, however, that the difference in loss of strength between the concrete in structures and that of the experiments would not be nearly as great as suggested by those comparative differences, because there is the partly compensating fact that structural members are generally exposed during construction to the drying effect of breezes and often to direct sunlight and a dry air, while the specimens were in a small closed room in which the air was comparatively moist. If, on the contrary, "the surface were kept wet for 7 days," it is believed that a large part of the loss on saturation would be avoided.

With regard to the values of deformations and sets, which Professor Hatt assumes were withheld, it can only be stated that circumstances prevented their determination in connection with the tests reported. The only additional information pertinent to the subject, which can be added, has reference to the suggestion of Mr. Worcester, that the actual strength of the specimens cured entirely in air "must have been rather lower than would naturally be expected of this quality of concrete." These cylindrical prisms averaged 1 760 lb. per sq. in. Though this is about one-tenth less than the compressive strength that may be fairly expected of good laboratory specimens of these materials

and proportions, they were actually somewhat stronger than the average of a large number of test prisms of similar concrete used in building construction and supposedly treated during curing in a way corresponding to that given to the material in the structures. Mr. Van Ornum.

The writer hesitated to publish the paper at all, because of the many supplementary questions remaining for investigation, the determination of which is necessary for a complete understanding of the phenomena involved. Nevertheless, he concluded that the field of further research would be made more definite and available to others if he presented the preliminary results already given. Of course, other percentage values will result from different experimental details, and the curve itself may be varied considerably in the same way. Yet it is believed that the broad, general fact of a significant loss in the strength of dry concrete when suddenly saturated and its gradual increase in strength to a final value considerably greater than the original one, when the saturation is prolonged, is a reality which should be definitely recognized in dealing with this important engineering material.

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Paper No. 1291

COAL PIERS ON THE ATLANTIC SEABOARD.*

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Scope of the Investigation.

On the Atlantic Seaboard from Maine to Florida there are four harbors which contain ports of importance where coal is discharged directly from commercial coal piers into the hatches of vessels. These harbors are New York, Philadelphia, Baltimore, and Norfolk.

There are 13 ports, embracing 29 coaling plants, in New York Harbor;
3 " " 11 " " " operation and 2 under construction in Philadelphia Harbor;
4 ports, embracing 8 coaling plants, in Baltimore Harbor;
3 " " 8 " " " operation and 2 under construction in Norfolk Harbor;

making a total of 23 ports, embracing 56 plants in operation and 4 under construction. In addition, the construction of new plants, at Baltimore, Philadelphia, and New York, is under consideration at the present time.

During the summer of 1912 the writer was retained to report on the facilities of these plants. With the assistance of his staff, each plant was examined, and investigations were made concerning its output, capacity, cargoes, type, tracks, grades, equipment, construction, and operation. With the consent of his client, the writer submits the

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substance of his investigations, with the hope that the information collated will be of interest to the members of the Society and of value to those concerned in the delivery of coal to vessels.

The principal data in connection with these piers are given in Tables 1, 2, 3, and 4, wherein the discharge is the maximum estimated capacity.

Output and Capacity.

The domestic and foreign shipments of coal from the various harbors during 1911 are given in Table 5.

Assuming that, on an average, a car contains 42 tons of coal, the tonnage in Table 5 indicates that about 1 050 000 cars were unloaded into boats of various kinds in 1911 from all the coaling ports.

The maximum capacities of the plants are given in Tables 1 to 4, and, as a general rule, are based on the largest number of cars handled in any one day, under favorable conditions of cargoes and deliveries. The average capacities are based on the deliveries obtained under ordinary conditions, where there is more or less delay due to the number and various sizes and kinds of cargoes, and approximate two-thirds of the maximum capacities.

During any of the past 8 years, the ratio of the actual maximum output to the estimated maximum capacities, as indicated in Tables 1 to 4, is about:

51% in New York Harbor;
29% " Philadelphia Harbor;
31% " Baltimore Harbor;
30% " Norfolk Harbor.

If all the ports had worked to their average estimated capacities, they could have discharged their output of 1911 in the following times:

New York Harbor	222	days.
Philadelphia Harbor	185	"
Baltimore Harbor	195	"
Norfolk Harbor	141	"

It will be noted that in New York Harbor the maximum output for any one of the past 8 years developed 51% of the maximum estimated capacity, and Philadelphia, Baltimore, and Norfolk only deliv-

ered about 30 per cent. This apparently better showing in New York Harbor can be explained by the fact that in that harbor there is an almost continuous run of cargoes varying from 50 to 3 500 tons for local consumption, and in other harbors the local consumption from the piers is not nearly as great; also, the cargoes are generally larger and of less frequency.

TABLE 1.—DATA CONCERNING

Railway or operator.	Location.	Type.
Eastern Coal Co. (P. R. R)	South Amboy, N. J	Loco. Incline A4
" " "	"	McMyler B2
" " "	"	B2
" " "	"	Grade A2
L. V. R. R	Perth Amboy, N. J	Grade and Gravity A8
"	"	" " A8
P. & R. Ry	Port Reading, N. J	Loco. Incline A4
"	"	" " A6
"	"	" " A6
C. R. R. of N. J	Elizabeth Port, N. J	" " A4
" "	"	" " A4
" "	"	" " A4
Wilkesbarre C. Co. (C. R. R. of N J.)	Port Johnson, N. J	" " A6
B. & O. R. R. Co	St. George, Staten Island	McMyler B2
"	"	Loco. Incline A4
"	"	" " A4
P. R. R	Port Greenville, N. J	" " A4
North River Co. (C. R. R. of N. J.)	Port Liberty, N. J	" " A6
Burns Bros., "	"	" " A4
Berwind-White Co. (P. R. R.)	Harsimus Cove, N. J	Grade A2
D., L. & W. R. R	Hoboken, N. J	Power Incline A8
"	"	McMyler B2
"	"	" B2
"	"	Endless Chain Conv'r. B5
D. & H	Weehawken, N. J	Power Incline A8
O. & W. R. R	Guttenberg, N. J	Loco. Incline A4
"	"	" " A4
Erie R. R	Edgewater, N. J	Power Incline A8
"	"	" " A8

* Maximum capacity of pier is 140 cars of

On the basis of the estimated maximum capacities of the various piers, the percentages of coal furnished by each of the roads serving would be about as given in Table 6.

From the fact that the actual annual output of these ports represents only from 141 to 222 working days under average capacity for the four harbors, it may be reasonably inferred that the existing

PIERS IN NEW YORK HARBOR.

Pier No.	Year built.	Length, in feet.	Height, in Feet.		Number of Tracks.				Grades. Percentage.				Discharge.				
			Shore.	Sea.	Appr.	Incl.	Deck.	Ret.	Appr.	Incl.	Deck.	Ret.	Sides.	Chutes.	Cars per 10 hours.		Total.
															Anthr.	Bitum.	
1	..	1 800	17	35	..	2	4	..	..	1.0	1.0	..	1	16	200	....	200
2	'09	....	..	..	5	1	..	1	1.6	12.0	..	1.6	1	1	....	280	280
3	'11	....	..	..	3	1	..	1	1.6	12.0	..	1.6	1	1	100*	200*	300
4	..	900	25	25	6	..	4	2	0	0	0	0	2	18	....	120	120
1	..	850	30	24	4	..	4	1	0.6	..	0.6	1.0	2	20	160	....	160
2	..	750	26	20	4	..	4	1	0.6	..	0.6	1.0	2	20	140	....	140
1	'99	550	12	18	..	1	3	..	..	1.0	1.0	..	2	16	60	....	60
2	'93	800	35	16	..	2	4	2	..	3.0	1.5	3.0	2	24	....	160	160
3	'06	900	42	24	..	2	4	2	..	3.0	1.5	3.0	2	36	....	180	180
1	..	500	15	20	..	1	3	..	..	1.0	1.0	..	1	4	60	....	60
2	..	500	15	20	..	2	3	..	..	1.0	1.0	..	2	10	80	....	80
3	'06	900	26	40	..	4	4	..	..	1.5	1.5	..	2	24	190	10	200
..	..	1 500	33	18	..	2	4	2	..	3.0	1.5	3.0	2	21	250	....	250
1	'05	....	..	..	1	1	..	1	2.0	12.0	..	1.5	1	1	....	200	200
2	..	400	34	34	..	1	3	..	..	3.2	0	..	2	14	....	60	60
3	..	330	32	32	..	1	3	..	..	3.5	0	..	2	6	....	40	40
..	..	400	29	35	..	2	3	..	..	2.5	1.5	..	2	14	200	....	200
1	..	1 700	29	20	..	2	2	1	..	2.5	1.5	2.5	2	18	170	....	170
2	..	1 600	30	30	..	2	4	..	..	3.0	0	..	2	6	20	100	120
..	..	1 750	30	30	1	..	3	..	0	..	0	..	1	20	....	300	300
1	..	1 200	38	25	..	2	2	1	..	16.0	1.0	1.0	1	5	20	....	20
2	'03	1 280	5	5	..	1	..	1		See	details.		1	1	200	....	200
3	'03	1 280	5	5	..	1	..	1		"	"		1	1	200	....	200
4	'03	....	5	5	..	..	..	..		"	"		2	18	10	....	10
..	'91	1 080	36	26	..	2	4	1	..	16.0	1.3	1.5	2	32	160	....	160
1	..	600	29	35	..	2	4	..	..	1.5	1.0	..	2	12	120	....	120
2	..	600	29	35	..	2	4	..	..	1.5	1.0	..	2	11	120	....	120
1	'94	850	40	30	..	2	4	1	..	16.0	1.5	2.0	2	12	120	....	120
2	'94	850	40	30	..	2	4	1	..	16.0	1.5	2.0	2	12	120	....	120
															2 700	1 650	4 350

anthracite, or 300 cars of bituminous, in 10 hours.

TABLE 2.—Data Concerning

Railway or operator.	Location.	Type.	Pier No. *	Year built.	Length, in feet.	Height, in Feet. Shore.	Sea.
Pennsylvania R. R.	Greenwich, Phila	Loco. Incline A4	1	'75	494	21	25
" "	" "	" " A4	2	'75	494	30	34
" "	" "	McMyler † B2	3	'13	800	14	14
" "	" "	Loco. Incl. A4	4	'75	641	28	42
" "	" "	Power Incl. A8	6	'01	735	72	66
B. & O. R. R. Co.	Jackson St., Phila.	Loco. Incline A6	1	'93	600	37	33
Phila. & Reading Ry.	Port Richmond, Phila.	Grade App. & Ret. A2	4	'40	188	21	21
" " "	" " "	" " " A2	8	'60	348	22	27
" " "	" " "	" " " A2	10	'60	239	20	22
" " "	" " "	Loco. Incl. A4	11	'97	760	42	51
" " "	" " "	Grade App. & Ret. A2	12	'65	201	21	22
" " "	" " "	Loco. Incl. A6	16	'92	714	38	30
" " "	" " "	" " †. A4	18	'12	765	54	62

* Includes maximum capacities of piers under construction.

TABLE 3.—Data Concerning

Railway or operator.	Location.	Type.	Pier No.	Year built.	Length, in feet.
B. & O. R. R. Co.	Curtis Bay, Balto.	Loco. Incline A6	..	'00	800
West. Maryland Ry	Port Covington, Balto.	Power Incline A8	..	'05	785
Pennsylvania R. R.	Clinton St. (Canton)	Loco. Incline A6	..	'87	790
" "	Foot Potomac St. (Canton).	" " A4	3	'87	398
" "	" " " "	" " A4	4	'87	392
Georges Creek Coal Co., B. & O. R. R.	Locust Point, Balto.	Loco. Incline A4	..	'87	248
B. & O. R. R. Co.	" " "	" " A4	..	'92	400
Merchants Coal Co., B & O. R. R.	" " "	" " A4	..	'87	150

PIERS IN PHILADELPHIA HARBOR.

Number of Tracks.				Grades, Percentage.				Discharge.				
										Cars per 10 hr.		
Appr.	Incl.	Deck.	Ret.	Appr.	Incl.	Deck.	Ret.	Sides.	Chutes.	Anthr.	Bitum.	Total.
1	3 & 2	3	..	....	+ 1.5	+ 0.78	......	1	10	130	....	130
1	3	4	..	....	+ 2.0	+ 0.87	......	2	19	....	300	300
4	1	1	1	− 1.8	+ 11.0	0.0	− 10 & 1	1	1	....	....	300
1	3	4	..	....	+ 2.25	+ 0.65	......	2	25	....	300	300
5	1	3 & 2	1	− 1.0	+ 17.5	− 1.0	− 2.8	2	40	....	....	300
1	1	2	1	0	+ 2.5	− 0.80	− 1.8	2	34	....	250	250
3	....	2	..	0	....	0	......	2	4	....	....	60
3	....	2	..	0	....	+ 1.36	......	2	11	....	100	100
3	....	3	..	0	....	+ 0.70	......	2	4	....	....	120
..	4	4	..	....	+ 1.25	+ 1.25	......	2	26	....	....	220
3	....	3	..	0	....	+ 0.58	......	2	6	60	....	60
4	2	2	2	0	+ 2.95	− 1.39	− 1.39	2	20	....	....	140
6	6	4	..	0	+ 2.08	+ 1.30	......	2	34	....	....	360
												*2 640

† Under construction.

PIERS IN BALTIMORE HARBOR.

Height, in Feet.		Number of Tracks.				Grades, Percentage.				Discharge.				
												Cars per 10 hr.		
Shore.	Sea.	Appr.	Incl.	Deck.	Ret.	Appr.	Incl.	Deck.	Ret.	Sides.	Chutes.	Anthr.	Bitum.	Total.
65	59	1	1 & 2	2	2 & 1	0	+ 2.0	− 1.0	− 2.0	2	48	..	300	300
67	60	1	1	2	1	− 2.0	+ 22.0	− 1.5	− 5.0 & − 3.0	2	40	..	250	250
42.2	38.6	1	2	2	1	+ 0.30	+ 2.0	− 0.50	− 0.47	2	24	..	120	120
19	21	..	3 & 1	3	..	..	+ 2.5	+ 0.50	....	2	10	80	....	80
27.5	29.5	1	2 & 5	5	..	+ 1.0	+ 2.5	+ 0.50	....	2	12	140	....	140
28	28	1	3	3	..	0	+ 5.0	0	....	2	10	..	60	60
35	35	1	1 & 3	3	..	0	+ 5.0	0	....	2	9	..	100	100
26	26	1	1 & 3	3	..	0	+ 5.0	0	....	2	4	40	....	40
												260	830	1 090

TABLE 4.—DATA CONCERNING

Railway or operator.	Location.	Type.	Pier No.	Year built.	Length, in feet.	Height, in Feet. Shore.	Sea.
Norfolk & Western Ry.	Lamberts Pt., Va.....	Loco. Incline......A6	1	'85	869	48.2	43.5
" " "	" " "	" " A6	2	'92	797	47	41.5
" " "	" " "	Power Incline......A8	3	'01	867	77.1	70.4
" " "	" " †"	Combination.......C1	4	'13	1 200	91.5	91.5
Virginian Ry. Co......	Sewells Pt., Va........	Combination.......C2	1	'09	1 000	76.1	69.2
Chesa. & Ohio Ry.....	Newport News, Va....	Loco. Incline A6	2	'87	345	41.5	39.5
" " ".....	" " ".....	" " A6	3	'82	790	38.5	34
" " ".....	" " †".....	Combination.......C1	9	'13	1 200	91.4	91.4
" " ".....	" " ".....	Loco. Incline A6	10	'00	450	51	48.5
" " ".....	" " ".....	" " A6	12	'07	850	59	55

* Includes capacities of piers under construction.

TABLE 5.—SHIPMENTS OF COAL, IN GROSS TONS.

Harbor.	Anthracite.	Bituminous.	Totals.
New York..........................	14 651 401	10 749 988	25 401 389
Philadelphia........................	2 197 750	4 856 626	7 054 376
Baltimore..............................	257 025	4 002 809	4 259 834
Norfolk................................	....	7 376 925	7 376 925
Totals....................	17 106 176	26 986 348	44 092 524

ports are capable of supplying greater demands for coal than now exist; but, of course, it must be remembered that a pier which supplies a succession of small cargoes, and especially to boats not under their own power control, will not be able to work to the same capacity as in cases where there will be the same continuous run of large cargoes regularly, and, though some of the ports may be hard pressed at times to make deliveries satisfactory to urgent demands, there are other ports which are more or less idle a great part of the time, and all the piers are idle some of the time, due to lack of vessels. The necessity of being in a position to furnish large cargoes in a short time requires, in a number of the ports, the construction of piers of greater capacity than they will be required to develop, except at intervals.

PIERS IN NORFOLK HARBOR.

Number of Tracks.				Grades, Percentage.				Discharge.				
										Cars per 10 hr.		
Appr.	Incl.	Deck.	Ret.	Appr.	Incl.	Deck.	Ret.	Sides.	Chutes.	Anthr.	Bitum.	Total.
1	1	2	1	−0.05	+2.3	−0.66	−2.5	2	22	..	150	150
1	1	2	1	−0.05	+2.3	−0.76	−2.4	2	27	..	150	150
5	1	2	1	−0.75 to −1.33	+25.0	−1.33 to −0.67	−2.83	2	60	..	400	400
					See sketch.			2	62	..	600	600
					See sketch.			2	62	..	300	300
1	1 & 2	2	1	0	+1.0 & 2.0	−0.63	−3.0 & −2.5	2	9	..	150	150
1	1	2	1	0	+1.0	−0.59	−0.59	2	28	..	250	250
					See sketch.			2	66	..	600	600
1	1 & 2	2	1	0	+2.0	−0.65	−2.2	2	20	..	200	200
1	1 & 2	2	1	0	+2.0	−0.65	−2.2 & −1.0	2	39	..	380	380
											*3 180	3 180

† Under construction.

TABLE 6.—PERCENTAGES OF COAL FURNISHED BY RAILROADS.

Railroad.	Harbors.				Total for all harbors.
	New York.	Philadelphia.	Baltimore.	Norfolk.	
Pennsylvania	32.0	53.2	31.0	....	30.0
Central of New Jersey	20.0	....	....	....	9.0
Delaware, Lackawanna and Western	10.0	....	....	....	4.5
Philadelphia and Reading	9.0	34.5	....	....	12.0
Baltimore and Ohio	7.0	12.3	46.0	....	11.0
Lehigh Valley	7.0	....	....	....	3.0
Erie Railroad	5.5	....	....	....	2.5
Ontario and Western	5.5	....	....	....	2.5
Delaware and Hudson	4.0	....	....	....	2.0
Western Maryland	....	....	23.0	....	3.0
Chesapeake and Ohio	....	....	....	49.5	10.0
Norfolk and Western	....	....	....	35.5	7.5
Virginian	....	....	....	15.0	3.0
Totals	100.0	100.0	100.0	100.0	100.0

The capacity of many of the piers is also restricted by lack of yard facilities, or by lack of proper adaptation for the kind of cargoes handled, the height, in a number of cases, being insufficient to permit of a free discharge of coal from the pockets to the vessels. The piers having the largest estimated maximum capacities per day of 10 hours in the various harbors are given in Table 7.

TABLE 7.—PIERS IN EACH HARBOR HAVING THE LARGEST ESTIMATED MAXIMUM CAPACITY.

Harbor.	Railroad.	Piers.	Location.	Type.	Number of cars.
New York...	P. R. R........	No. 3.	South Amboy..	McMyler dumper, Type B2....	300
	P. R. R........	....	Harsimus Cove	Grade approach and return, Type A2....................	300
Philadelphia.	P. R. R........	No. 6.	Greenwich.....	Power incline, Type A8........	350
	P. R. R........	Nos. 2 and 4.	"	Locomotive incline, Type A4.	300
Baltimore....	B. & O.........	....	Curtis Bay....	Locomotive incline, Type A6.	300
Norfolk......	N. & W........	No. 3.	Lamberts Point........	Power incline, Type A8........	400
	C. & O.........	No. 12.	Newport News	Locomotive incline, Type A6.	380
	Virginian R.R.	....	Sewells Point..	Combination, Type C2.......	300
	New combination plants, Type C1, under construction, one for N. & W. at Lamberts Point and one for C. & O. at Newport News, will have a maximum capacity of..............................				600

CARGOES.

In New York Harbor coal is delivered mostly to lighters and barges up to 3 500 tons capacity, there being but few sea-going vessels which are coaled directly from the piers, either in bunkers or in cargoes. These are usually loaded from lighters. Some piers are prepared to furnish coal in from 5- to 10-ton lots, but most of the cargoes delivered are not less than 50 tons.

In Philadelphia Harbor coal is delivered to local tugs, barges, schooners, scows, tramps, and colliers, the greater portion being in barges for domestic use, cargoes running up to 3 500 tons, and to a number of tramps of from 5 000 to 7 000 tons, with an occasional collier of 12 500 tons.

In Baltimore Harbor coal is delivered to small bay craft up to 50 tons, coastwise sailing vessels with cargoes of from 1 500 to 6 000 tons, barges up to 7 500 tons, tramps from 5 000 to 7 000 tons, and colliers of 12 500 tons. A large part of this coal goes to New England and to foreign countries.

In Norfolk Harbor cargoes are delivered to boats of about the same class as in Baltimore Harbor, but some of the Norfolk piers are better adapted for furnishing coal to the largest Government colliers.

The two largest Government colliers, the *Cyclops* and *Neptune*, have a length of 542 ft., beam 65 ft., hatches 32 ft. athwartships, 13 ft. fore and aft; the top of the forward cargo hatch when the boat is in ballast trim is about 30 ft. above the water level, and the aft cargo hatch 25 ft., the bunker hatches being about 38½ ft. When out of

ballast trim, a condition which may occur rarely, the aft bunker hatches may be about 43 ft. above the water line. Their main draft when loaded is 27 ft. 7 in. On account of the steel cranes and derricks on the decks of these boats, it is somewhat difficult to coal them.

The largest coastwise steamer of the New England Coal and Coke Company is the *Malden,* which has a length of 406 ft. over all, beam 54 ft. 6 in., ten cargo hatches 30 ft. athwartships and 15 ft. fore and aft, one bunker hatch 24 ft. athwartships, 7 ft. fore and aft. The distance from the top of the cargo hatches to the water, when the boat is in ballast trim, varies from 26 to 32 ft., and the bunker hatch is 32 ft. It is probable that in the future this company will build colliers up to 550 ft. in length, with from 65 to 70 ft. beam.

Channels and Tidal Variations.

The present depths of the channels leading to the various ports in the four harbors are given in Table 8.

Classification of Coal Plants.

The coal piers constructed for the purpose of supplying coal to vessels in the harbors of the Atlantic Seaboard may be divided into the three general classes:

(A) Gravity Plants,
(B) Mechanical Plants,
(C) Combination Plants.

A.—Gravity Plants.

This name is generally given to coal trestles which are equipped with pockets or bins for receiving the coal from the usual cars, and with chutes for discharging it into vessels moored at the piers. Although a number of these plants have such grades as will permit the loaded cars to drift over the deck tracks and the empties to return to the yards by gravity, there are also a number where the grades are so light that power of some kind is required for the movement of the loads over the deck and of the empties to the yards; but, no matter what the grades or how the cars are moved on the deck and on the return tracks, the pier is classed as a gravity plant if the coal is discharged directly into the pockets or chutes from the regular coal cars on the tracks on top of the pier.

TABLE 8.—DEPTHS OF CHANNELS, AND TIDAL VARIATIONS.

Harbor.	Port.	Channels, etc.	Depth, in feet.	Tidal Variations, in Feet. Mean.	Tidal Variations, in Feet. Maximum.
New York...	South Amboy.....	Raritan Bay..........	21	4.5	6.3
	Perth Amboy.....	" "	21		
	Port Reading....	Arthur Kill..........	21		
		Staten Island Sound and Kill van Kull..........	16		
	Elizabeth Port..	Kill van Kull..........	16		
		Arthur Kill and Staten Island Sound..........	21		
	Port Johnson......	Kill van Kull..........	26		
	St. George.......	Upper and Lower Bays..........	40		
	Greenville.......	Upper and Lower Bays..........	40		
		Local to pier..........	18		
	Port Liberty......	Upper and Lower Bays and Hudson River..........	40		
	Harsimus Cove....	Upper and Lower Bays and Hudson River..........	40		
	Hoboken..........	Upper and Lower Bays and Hudson River..........	40		
	Weehawken.......	Upper and Lower Bays and Hudson River..........	40		
	Guttenberg	Upper and Lower Bays and Hudson River..........	40		
	Edgewater........	Upper and Lower Bays and Hudson River..........	40		
Philadelphia.	Greenwich.........	Delaware River 27 ft., being dredged to:..........	35	5.3	5.8
	Jackson St.......	Delaware River 27 ft., being dredged to:..........	35		
	Port Richmond....	Delaware River 27 ft., being dredged to:..........	35		
Baltimore...	Curtis Bay.........	Local channel 31 ft., Harbor channel..........	35	1.2	6.0
	Port Covington....	Local channel 25½ ft., Harbor channel..........	35		
	Canton.........	Clinton St. 30 ft., Harbor channel.	35		
		Piers 3 and 4 22 ft., Harbor channel..........	35		
	Locust Point......	Local channel 27 ft., Harbor channel..........	35		
Norfolk......	Lamberts Point...	Hampton Roads and Elizabeth River..........	35	2.7	3.1
	Sewells Point.....	Hampton Roads..........	35		
	Newport News....	Hampton Roads and James River.	35		

The general class of gravity plants embraces nine types, as shown by Figs. 1 and 2.

Type A1.—Gravity Approach and Return.—The loads are delivered to the pier deck, spotted over the pockets, and the empties returned to the yards, all by gravity. This is the ideal type of gravity plant; but locations where the land back of the dock is at a sufficient elevation for a gravity lead to the pier deck, and where the empty yard can be constructed at an elevation lower than the deck of the pier, are not usual in harbors. As a matter of fact, the writer has been unable to

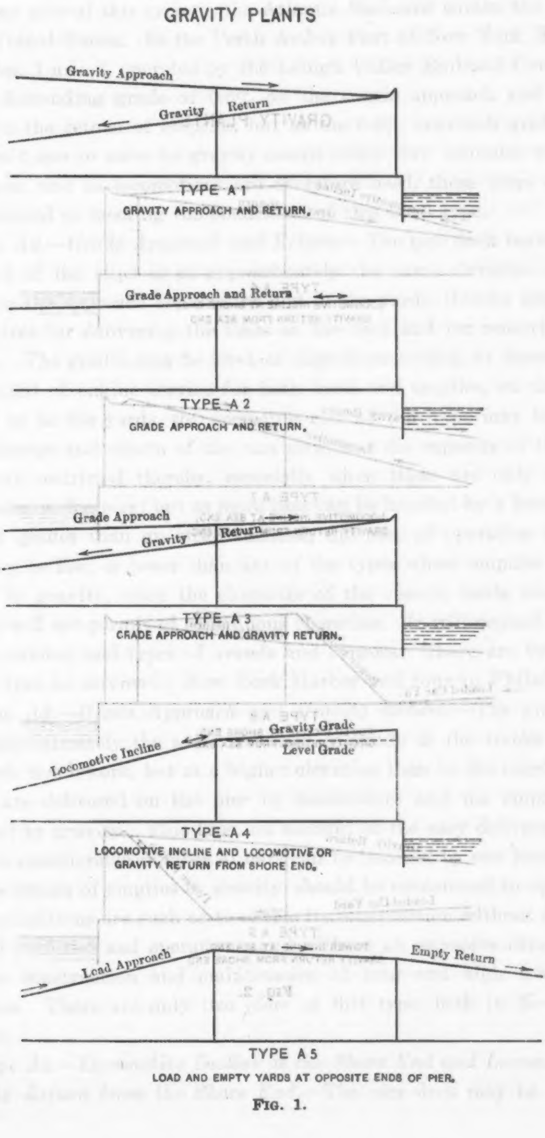

GRAVITY PLANTS
Gravity Approach
Gravity
Return
TYPE A 1
GRAVITY APPROACH AND RETURN.
Grade Approach and Return
TYPE A 2
GRADE APPROACH AND RETURN.
Grade Approach
Gravity
Return
TYPE A 3
GRADE APPROACH AND GRAVITY RETURN.
Gravity Grade
Level Grade
Locomotive Incline
TYPE A 4
LOCOMOTIVE INCLINE AND LOCOMOTIVE OR GRAVITY RETURN FROM SHORE END.
Load Approach
Empty Return
TYPE A 5
LOAD AND EMPTY YARDS AT OPPOSITE ENDS OF PIER.

FIG. 1.

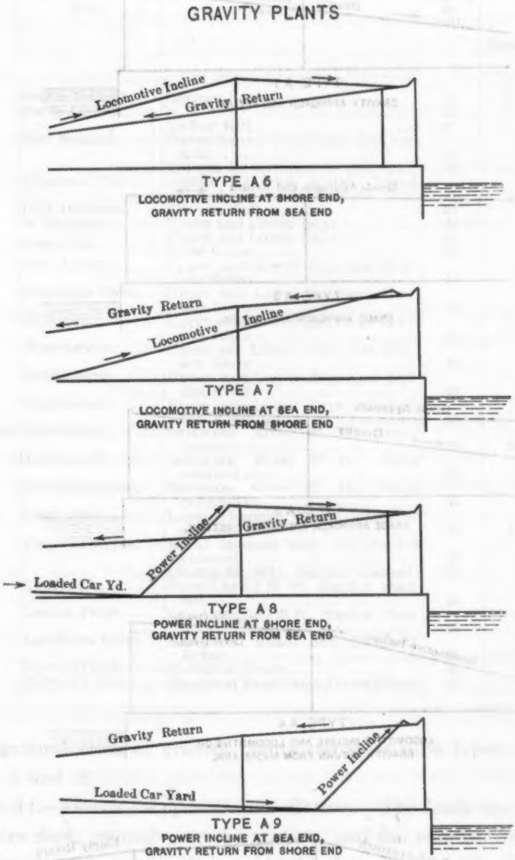

GRAVITY PLANTS
Locomotive Incline
Gravity Return
TYPE A 6
LOCOMOTIVE INCLINE AT SHORE END,
GRAVITY RETURN FROM SEA END
Gravity Return
Locomotive
Incline
TYPE A 7
LOCOMOTIVE INCLINE AT SEA END,
GRAVITY RETURN FROM SHORE END
Power Incline
Gravity Return
Loaded Car Yd.
TYPE A 8
POWER INCLINE AT SHORE END,
GRAVITY RETURN FROM SEA END
Gravity Return
Power Incline
Loaded Car Yard
TYPE A 9
POWER INCLINE AT SEA END,
GRAVITY RETURN FROM SHORE END

Fig. 2.

locate any pier of this type on the Atlantic Seaboard within the limits of the United States. In the Perth Amboy Port of New York Harbor, Piers Nos. 1 and 2, operated by the Lehigh Valley Railroad Company, have a descending grade of 0.6% on the trestle approach and a 1% grade for the return of empties, but as the 0.6% approach grade will not permit cars to move by gravity except under very favorable weather conditions, and as locomotives are therefore used, these piers cannot be considered as meeting the conditions of this ideal type.

Type A2.—Grade Approach and Return.—The pier deck beyond the bulkhead of the slips is at approximately the same elevation as the tracks on the approach trestlework and in the yards, thereby requiring locomotives for delivering the loads on the deck and for removing the empties. The grades may be level, or slightly ascending, or descending. On account of engine service for both loads and empties, on the deck as well as in the yards, the operating cost of this type may be high, the discharge and return of the cars slow, and the capacity of the pier somewhat restricted thereby, especially when there are only one or two tracks on the pier; but as more cars can be handled by a locomotive on easy grades than on steep inclines, the cost of operation of this type may be low, or lower than any of the types where empties are returned by gravity, when the character of the vessels, loads, and their cargoes will not permit of continuous operation. It will depend largely on the number and types of vessels and cargoes. There are two piers of this type in service in New York Harbor and four in Philadelphia.

Type A3.—Grade Approach and Gravity Return.—The pier deck is at approximately the same elevation or grade as the tracks on the approach trestlework, but at a higher elevation than in the empty yard. Loads are delivered on the pier by locomotives and the empties are returned by gravity. This type, on account of the easy delivery grade, where a considerable number of cars can be handled by one locomotive, and the return of empties by gravity, should be economical in operation where conditions are such as to enable its construction without sacrifice of yard facilities and operations, and without an excessive expenditure for the construction and maintenance of long and high trestle approaches. There are only two piers of this type, both in New York Harbor.

Type A4.—Locomotive Incline at the Shore End and Locomotive or Gravity Return from the Shore End.—The pier deck may be level or

have a slight grade. The loads are pushed up the incline by a locomotive and are then handled and the empties removed by the locomotive, or the empties are spotted by gravity or by using pinch-bars and then returned by gravity down the delivery incline, depending on the grades. This is a simple type of so-called gravity pier, and is used where the width of right of way is contracted and local conditions are such as to necessitate the construction of a joint yard for loads and empties within a restricted space. The empties may be returned over the delivery tracks, for piers where the output is small, or over independent tracks, where the output is large and the right of way will permit the additional width necessary for such tracks. There are eleven piers of this type in New York Harbor, five in Philadelphia, and five in Baltimore, making a total of twenty-one now in operation.

Type A5.—Load and Empty Yards at Opposite Ends of Pier.—The loads are pushed from the yard up the incline at one end of the pier, and the empties are returned to the yard at the other end, the entire movement of cars being in one direction. This type may be used along the banks of rivers or canals, where the yards and tracks have to be parallel to the stream, and is used for locomotive coaling stations, but, so far as the writer has been able to discover, there are no piers of this type on the Atlantic Seaboard.

Type A6.—Locomotive Incline at Shore End, Gravity Return from Sea End.—Loads are pushed up the incline to the deck by a locomotive, then drift by gravity over the pockets, and, when unloaded, drop by gravity to a switchback or turn-table, where the movement is reversed to the empty yard. The empty return tracks, being independent of the approach tracks, may be graded so as to reach an empty yard adjoining the load yard or at almost any convenient location, without impeding operations. All things considered, this type is perhaps the most practical and economical of the gravity plants, both as to cost and operation, and is sufficiently flexible to meet a diversity of local conditions. There are four piers of this type in New York Harbor, two in Philadelphia, two in Baltimore, and six in Norfolk, making a total of fourteen now in operation. The Baltimore and Ohio pier at Curtis Bay, Baltimore Harbor, is a well-known example.

Type A7.—Locomotive Incline at Sea End, Gravity Return from Shore End.—This is simply a modification of Type A6. The lead for the yard tracks is nearer the water and the empty yard is farther back

than in Type A6. The deck of the pier has an ascending grade toward the sea end, thereby making the highest part of the deck at the place where it is of most service. There are no piers of this type in operation, so far as can be learned by the writer, but the advantages of the type in certain locations make it worthy of consideration.

Type A8.—Power Incline at Shore End, Gravity Return from Sea End.—The pier has a down grade toward the sea. Cars are pulled up the incline by a cable and stationary engine, or are pushed up by a small car or barney attached to the end of a cable. Otherwise the operation is the same as in the locomotive incline, Type A6. The power inclines, with their steep grades, are adaptable to locations where the space at the approach to the pier is not sufficient for a locomotive incline. The delivery track from the loaded yard is usually graded downward so as to feed the cars to the foot of the incline by gravity. There are four piers of this type in New York Harbor, one in Philadelphia, one in Baltimore, and one in Norfolk, making a total of seven, exclusive of a similar plant in combination with a mechanical plant, which is embraced under another classification.

Type A9.—Power Incline at Sea End, Gravity Return from Shore End.—The pier has an up grade toward the sea, and is simply a modification of Type A8. The highest part of the deck is at the sea end, where the highest chutes properly belong. This type, when vessels are loaded from only one side of the pier, is well adapted to localities where the load yard is close to the pier. When piers deliver coal to vessels on both sides, the delivery track can be placed in the middle and the deck tracks can be carried across the entire width of the pier, except for the space required for the incline through the deck, thereby requiring less width of pier at the shore end than for Type A8. There are no piers of this type in operation on the Atlantic Seaboard, but it is worthy of consideration in favorable locations.

B.—Mechanical Plants.

Coaling plants which are equipped with car-dumping machinery, elevators, conveyors, etc., for delivering coal to vessels without the use of high and long trestles, are classed as mechanical. The car-dumping machines pick up the loaded cars and dump their contents on an apron, from which it flows into vertical chutes which are adjustable and kept full of coal so as to reduce the fall and breakage. These

machines vary somewhat in design, and are manufactured by the McMyler Manufacturing Company, the Wellman-Seaver-Morgan Company, the Brown Hoisting and Conveying Machine Company, all of Cleveland, Ohio. A description of the several types of mechanical plants (Figs. 3 and 4) follows:

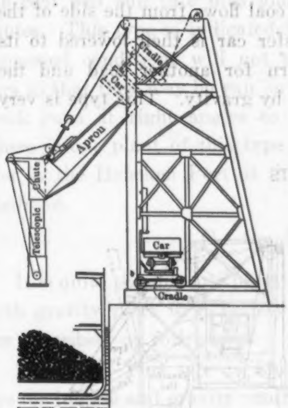
B1.—Car-Dumping Machine at Grade.—The loaded car is delivered near the cradle of the machine, which is at grade or only a slight elevation above it, and is placed or hauled on the cradle. It is then lifted vertically and dumped into chutes by turning it over sideways. The car is then lowered, pushed off the cradle by the next loaded car entering, and hauled away. The range of delivery to vessels is confined within the sweep of the chutes, and this necessitates moving the vessels while being loaded. This type is adapted to locations where space is restricted to such an extent that the gravity movement of empty cars is impracticable, but there is no plant of this particular type in operation on the Atlantic Seaboard.

B2.—Car-Dumping Machine with Power Incline.—The loaded car is delivered by gravity or locomotive to a barney or cable connection which moves up a steep inclined plane to the cradle of the machine. It is then dumped into chutes by turning it over sideways, and the empty car is returned to the yard by gravity. This type requires more space than B1, and, as the cars are delivered at an elevation, the vertical lift of the machine need not be as great. When space is available, this plant is more economical of operation than Type B1. There are five plants in New York Harbor and one in Philadelphia, a total of six on the Atlantic Seaboard, exclusive of those used in combination with gravity piers and included in Class C.

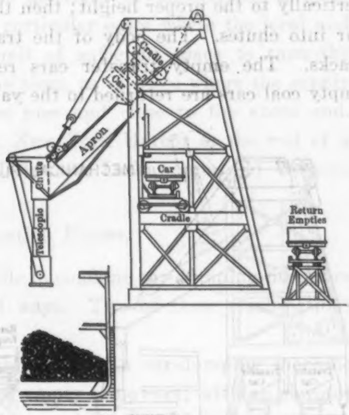
B3.—Car-Dumping Machine with Conveyors.—The machine turns the car over sideways and dumps the coal into conveyor buckets which carry it from the machine and lower it to the bottom of the hatch before dumping. The cars may be handled by locomotives or by barneys and gravity. It is claimed that this will reduce the breakage of coal to a minimum. There are no plants of this type on the Atlantic Seaboard, but one is in operation at Buffalo, N. Y.

B4.—Car-Dumping Machine with Movable Tower.—The loaded car is delivered by a locomotive or by gravity to a barney, which moves it up an inclined plane to the cradle of the machine. It is then dumped into a self-propelling transfer car having a sloping bottom and dis-

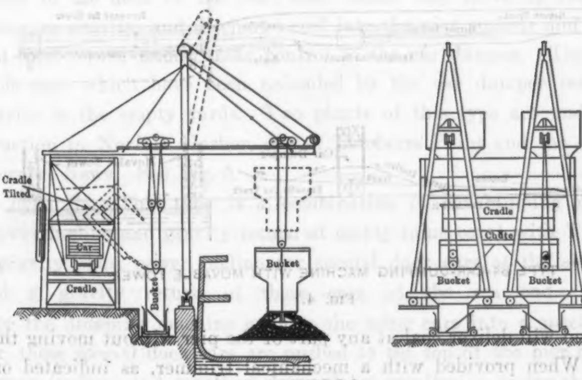
charge, which moves on to a movable tower. The tower moves on tracks running along the pier, and elevates the body of the transfer car vertically to the proper height; then the coal flows from the side of the car into chutes. The body of the transfer car is then lowered to its tracks. The empty transfer cars return for another load and the empty coal cars are returned to the yard by gravity. This type is very



TYPE B 1
CAR-DUMPING MACHINE AT GRADE



TYPE B 2
CAR-DUMPING MACHINE WITH BARNEY

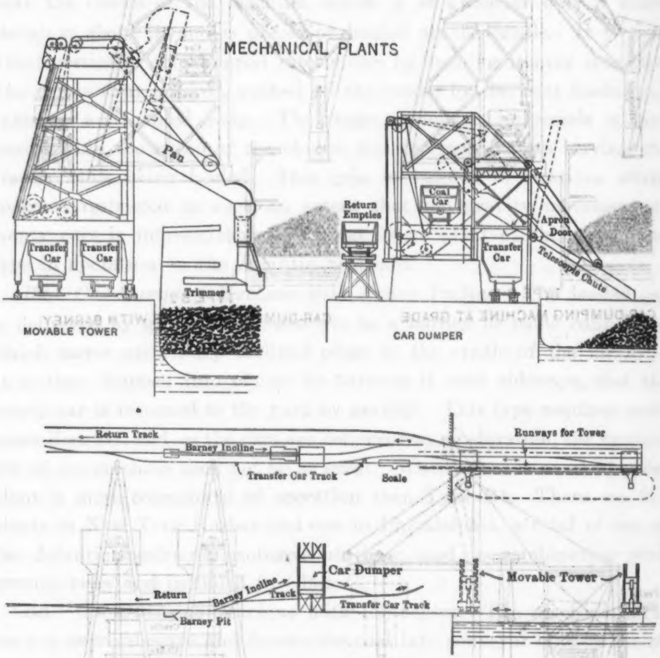


TYPE B 3
CAR-DUMPING MACHINE WITH CONVEYOR

FIG. 3.

elastic and will deliver coal at any part of the pier without moving the vessels. When provided with a mechanical trimmer, as indicated on the sketch (Fig. 1), the cost of trimming coal may be reduced considerably. There are as yet no plants of this type in operation on the Atlantic Seaboard, but a number of designs have been under consideration.

charge, which moves on to a movable tower. The tower moves on tracks running along the pier, and elevates the body of the transfer car vertically to the proper height; then the coal flows from the side of the car into chutes. The body of the transfer car is then lowered to its tracks. The empty transfer cars return for another load and the empty coal cars are returned to the yard by gravity. This type is very



TYPE B4-CAR-DUMPING MACHINE WITH MOVABLE TOWER

FIG. 4.

elastic, and will deliver coal at any part of the pier without moving the vessels. When provided with a mechanical trimmer, as indicated on the sketch (Fig. 4), the cost of trimming coal may be reduced considerably. There are as yet no plants of this type in operation on the Atlantic Seaboard, but a number of designs have been under consideration.

B5.—Endless Chain Conveyors.—The coal is dumped into a hopper and then elevated by an endless chain conveyor to such a height that the coal will run into bins and be delivered to the hatches through chutes. This type is indicated in particular cases where the local and congested conditions will not permit of sufficient space to turn the cars so that they may be run on the piers, as in cases where the supply track runs at right angles to the pier and close to the shore end. There is one plant of this type in New York Harbor at the end of a pier in the Hoboken Port of the Delaware, Lackawanna and Western Railroad.

C.—Combination Plants.

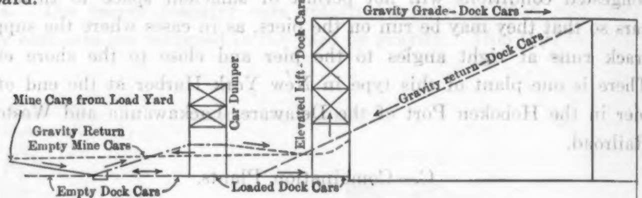
It is quite possible and practicable to combine car-dumping machines with gravity piers in a variety of ways. Two of these combinations are described, as follows:

Type C1.—This type is a combination of a car-dumping machine (power incline and gravity return of empty mine cars) with an elevator lift to a gravity pier deck at the shore end for special dock cars, and a gravity return of empty dock cars from the sea end. The mine cars are dumped by the machine into special dock cars which move by their own power, or by gravity, to the platform of the elevator, are then raised to the deck of the pier, over which they move by their own power, or gravity, and discharge coal into the pier pockets and chutes, and then return under brake control to the car dumper. The empty mine cars which have been unloaded by the car dumper return by gravity to the empty yards. Two plants of this type are under construction in Norfolk Harbor, one at Lamberts Point and the other at Newport News. See Fig. 5.

Type C2.—This type is a combination of car-dumping machine (power incline and gravity return of empty mine cars) with Type A8, a gravity plant power incline for special dock cars at the shore end and a gravity return of these cars at the sea end. In this type the dumping machine empties the mine cars into a special dock car, these special dock cars are pushed to the top of the pier over the barney incline, and are then dropped by gravity, or under their own control, to the pockets where the coal is delivered; they are then returned to the car dumper under brake control over the gravity return. The empty mine cars drift by gravity over the switchback to the empty

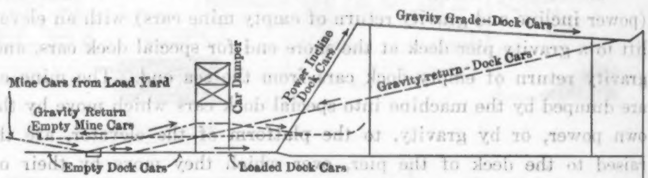
yards. There is one pier of this type in Norfolk Harbor at Sewells Point. See Fig. 5.

Table 9 shows the number of each class and type of plant now in operation or under construction in the harbors of the Atlantic Seaboard.



TYPE C 1

CAR-DUMPING MACHINE (POWER INCLINE AND GRAVITY RETURN OF EMPTY MINE CARS) WITH ELEVATOR LIFT TO PIER DECK AT SHORE END FOR SPECIAL DOCK CARS, AND GRAVITY RETURN OF EMPTY DOCK CARS FROM SEA END.



TYPE C 2

CAR-DUMPING MACHINE (POWER INCLINE AND GRAVITY RETURN OF EMPTY MINE CARS) WITH TYPE A 8, GRAVITY PLANT, POWER INCLINE FOR SPECIAL DOCK CARS AT SHORE END AND GRAVITY RETURN OF SPECIAL DOCK CARS FROM SEA END.

Fig. 5.

Dimensions.

The longest coal pier, measured from the bulkhead to the end of the slip, is Pier No. 1, Pennsylvania Railroad Company, at South Amboy, N. J., which has a length of 1 800 ft. The length of the Curtis Bay pier of the Baltimore and Ohio Railroad is 800 ft., of the Norfolk and Western pier at Lamberts Point, 867 ft., of the Virginian Railway pier at Sewells Point, 1 000 ft., of Pier No. 12 of the Chesapeake and Ohio Railway at Newport News, 850 ft., and of Pier No. 6 of the Pennsylvania Railroad at Greenwich, Pa., 735 ft. These represent the lengths of the slips. The two piers at Norfolk now under construction, one for the Norfolk and Western Railroad and one for the Chesapeake and Ohio Railroad, will have slips 1 200 ft. long.

TABLE 9.—NUMBER OF EACH CLASS AND TYPE OF PLANT IN OPERATION OR UNDER CONSTRUCTION IN THE HARBORS OF THE ATLANTIC SEABOARD.

Harbor.	Gravity Plants. Class A.										Mechanical Plants. Class B.						Combination Plants. Class C.		
	A1.	A2.	A3.	A4.	A5.	A6.	A7.	A8.	A9.	Total.	B1.	B2.	B3.	B4.	B5.	Total.	C1.	C2.	Total.
New York	..	2	2	11	..	4	..	4	..	23	..	5	..	..	1	6	..	..	..
Philadelphia	..	4	..	5*	..	2	..	1	..	12	..	1†	..	..	..	1	..	..	..
Baltimore	..	..	..	5	..	2	..	1	..	8	..	..	..	..	..	..	..	..	..
Norfolk	..	..	..	..	..	6	..	1	..	7	..	..	..	..	..	..	2†	1	3
Totals	0	6	2	21	0	14	0	7	0	50	0	6	0	0	1	7	2	1	3

* One plant under construction and four in service.

† Under construction.

The highest pier thus far built is the Norfolk and Western Pier No. 3, at Lamberts Point, which has a height of 70.4 ft. above the water near the sea end. The Curtis Bay pier of the Baltimore and Ohio Railroad has a height of 59 ft. near the sea end, the Virginian Railroad pier at Sewells Point has a height of 69.2 ft., and the two piers under construction at Lamberts Point and Newport News will have a height of about 91½ ft.

The heights and lengths of all piers on the Atlantic Seaboard are noted in Tables 1 to 4. The highest piers, namely, those under construction at Newport News and Lamberts Point, will have a maximum height of 47 ft. from the discharging end of the chute to the water, and a minimum of 5 ft.

The dimensions of the largest U. S. naval colliers and coastwise steamers carrying cargo coal are given under the heading, "Cargoes." In order to coal freely into coastwise boats, the maximum height of hatches being 32 ft., the height of piers should not be less than 65 ft. above mean tide, when the pier is not provided with storage bins, and, if bins are provided for storing one car of coal each, the height should be at least 70 ft. above mean tide. For coaling the largest Government colliers freely, the height should be at least 8 ft. greater than the foregoing, and to meet the possible conditions when the boat is out of ballast, the height should be at least 13 ft. greater.

Piers with heights 5 ft. less than those given will coal to the same class of boats, but not freely, because the chutes will be inclined at too flat an angle for the gravity flow of coal. A pier only 50 ft. high above mean tide can be made to deliver coal to some of the coastwise steamers with capacities up to 7 000 tons, but at extra expense for handling and trimming. As a general proposition, for efficient service to boats of all classes, the height of a pier should be from 33 to 43 ft. greater than the height of the highest hatches of vessels coaling at the ports, depending on the type of chute used and the storage capacity of the bins.

Tracks and Grades.

The tracks leading to piers and yards are usually arranged so as to require the least number of shifting operations. In some yards a large number of short tracks are required to sort cars with different kinds of coal, and when this is the case, the load yard, when practicable, consists of a number of short tracks, or a gridiron arrangement with a ladder at each end. For the empty-car yard, sorting is generally not necessary, and a few long tracks are used. The approach tracks to locomotive inclines usually have as great a length as the situation will permit, in order to enable locomotives to have a good run for overcoming the grade of the incline. The approach tracks leading to barney inclines have a down grade where the conditions will permit, in order that the cars may drift by gravity to the barney. Tracks on the piers are arranged in a number of ways, depending on the local conditions, the type of pier, and whether coaling is done from one or both sides.

The grades on locomotive inclines vary from 1 to 5 per cent. There are only three piers having grades of 5%, six with grades from 3 to 3½%, and twenty-six with grades less than 3 per cent. The conditions which affect the choice of the grade are: longitudinal space, capacity of the locomotive to be used, the required discharging capacity of the pier, and economy of operation and cost. A steep locomotive incline will cost less to construct, but usually more to maintain and operate, than an easy or moderate grade, and, as the delivery of the coal to the pier deck should keep pace with the discharge into vessels, for economical operation, the grade should not be such as to retard delivery. The selection of the grade on locomotive inclines is an important matter, and should be determined, not so much from what is

in use on other piers as by a study of the local conditions and requirements for each case, and when conditions permit, it should not exceed 3 per cent.

The grades on barney or cable inclines to gravity class piers vary from 16 to 25%, and on mechanical classes from 11 to 12 per cent. It will require less power and less expensive machinery to pull a car up a 12% grade than up a 25% grade, and though the steeper inclines are usually resorted to on account of limited longitudinal space, it is economy to use the easiest grade permitted by the local conditions—economy even in first cost—as the additional cost of the longer low-grade trestle is often offset by the cost of less expensive machinery, and there is less danger of accident. In mechanical classes of plants, as the inclines are required for relatively small height, it is not necessary to give them a very steep grade. A 16 to 18% grade on a gravity plant, and 10 to 12% on a mechanical plant, appear to be reasonable.

The grades on pier decks vary from 0.5 to 1.5 per cent. When the grade is less than 1%, a short, sharp grade is sometimes introduced to give an impetus to the cars at the head of the pier and at the switch-back. The object of a grade on the pier deck is to enable cars to move slowly by gravity. In the summer cars in good condition will run freely on an 0.8% grade without resort to pinch-bars. As coal is delivered to vessels during cold as well as warm weather, and in northern as well as southern climates, the selection of a grade for any pier is more or less a matter of compromise; grades from 1 to 1.3% give good results in New York, but grades from 0.67 to 0.76% appear to give equally good results in the South when headed by a sharp starting grade.

Grades for the return tracks vary from 0.6 to 5% for hand-brake control, but it is considered good practice to keep them less than 2% when practicable, and to ease them off as their lower ends approach the empty yard. The percentage of grade of the return track is governed to a considerable extent by the location of the empty yard.

All changes in grade are eased by vertical curves extending for a distance of from 35 to 50 ft., and even for a longer distance where circumstances warrant, depending on the degree of change and the length of grades. The number of tracks on approaches, including deck and empty return, with their grades, are given in Tables 1 to 4.

Switchbacks and Pivot Tables.

Switchbacks are intended to work automatically, and have grades and lengths proportioned so as to keep the empty cars moving by gravity. They are generally constructed with adjustable blocking, in order that the grades may be quickly adjusted during the different seasons of the year, as those having grades satisfactory for handling cars by gravity in hot weather will not operate well in cold weather and *vice versa.* The grades, etc., of a typical switchback are indicated on the sketch, Fig. 6.

In some cases, where longitudinal space will not permit the construction of a switchback, counterbalanced transfer or pivot tables are used. The normal position of the table is in line with the delivery track. Cars are run on the table by gravity, and the table moves by gravity to a connection in line and grade with the return track, the empty car then leaves the table by gravity, and the table is returned to its normal position by the counterweight. Pivot tables of this character are in use at Lamberts Point, Piers Nos. 1 and 2, in Norfolk, and at Port Covington and Clinton Street, in Baltimore. One man is required at each table to control the brake levers governing the movement of the table. The arrangement of a typical table is shown on the sketch, Fig. 6.

Wherever there is sufficient space for the construction of a properly designed switchback, it is preferable to a pivot table, as it has less mechanism to get out of order and is equally effective.

Equipment.

The equipment in connection with the gravity type of pier as herein discussed embraces bins, pockets, chutes, and power-incline mechanism, and, for all classes of piers, scales and thawing plants.

Bins and Pockets.—For the purposes of this paper, the following classification of bins and pockets on coal piers has been made:

Bins.—Hoppers or receptacles constructed for the purpose of storing coal temporarily or permanently;

Pockets.—Hoppers constructed to receive the coal from the cars and to deliver directly into the chutes without storage.

There are various forms of construction of bins and pockets; a general description of each follows.

Bins in most cases consist of partitions framed to timber bents of the approach trestle; they have wooden floors, and the coal is dropped from the car bottoms through openings in the deck. In some instances these bins are constructed with enclosed sides and with a hopper bottom, so that the coal may be reloaded into cars or loaded into wagons

TYPICAL SWITCHBACK AND PIVOT TABLE

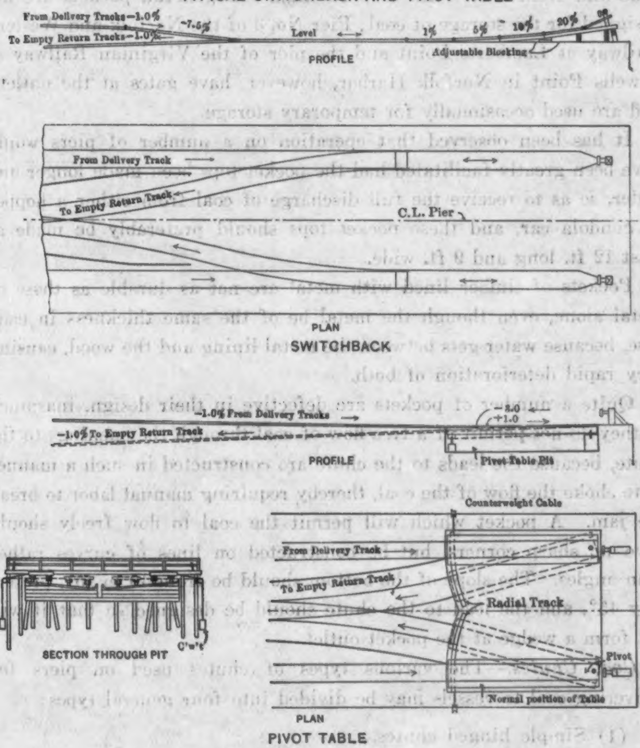


FIG. 6.

or boats through chutes. When bins have sides and hopper bottoms they are usually lined with sheet metal.

Pockets are ordinarily used to receive the coal as it drops from the hopper bottom of the car and convey it to the chutes. Pockets are located under the delivery tracks on the pier, and have their bottom outlet at the side of the pier from 8 to 12 ft. below the deck.

Most of the piers examined were equipped with timber pockets, metal lined, although the pier of the Virginian Railway at Sewells Point in Norfolk has metal pockets. The upper ends of the pockets are generally about 9 ft. long, and from 5 to 9 ft. wide, the sides and bottom tapering to an opening at the side of the pier about 4 ft. 6 in. wide and from 2 ft. 6 in. to 3 ft. high. In general, pockets are not designed for the storage of coal; Pier No. 3 of the Norfolk and Western Railway at Lamberts Point and the pier of the Virginian Railway at Sewells Point in Norfolk Harbor, however, have gates at the outlets, and are used occasionally for temporary storage.

It has been observed that operation on a number of piers would have been greatly facilitated had the pocket tops been made longer and wider, so as to receive the full discharge of coal from either a hopper or gondola car, and these pocket tops should preferably be made at least 12 ft. long and 9 ft. wide.

Pockets of timber lined with metal are not as durable as those of metal alone, even though the metal be of the same thickness in each case, because water gets between the metal lining and the wood, causing very rapid deterioration of both.

Quite a number of pockets are defective in their design, inasmuch as they do not permit of a free flow of coal through the openings to the chute, because the leads to the chute are constructed in such a manner as to choke the flow of the coal, thereby requiring manual labor to break the jam. A pocket which will permit the coal to flow freely should have no sharp corners, but be constructed on lines of curves rather than angles. The slope of the bottom should be from 40 to 45°, preferably 45°, and the lead to the chute should be designed so that it will not form a wedge at the pocket outlet.

Coal Chutes.—The various types of chutes used on piers for delivering coal to vessels may be divided into four general types:

(1) Simple hinged chutes,
(2) Hinged chutes adjustable at the upper end,
(3) Hinged chutes with telescopic leg at the upper end,
(4) Chutes with telescopic leg at the lower end.

Some of these types are patented, and have different trade names, according to the notions of the patentees, but the foregoing general classification will best serve the purpose of this paper. Fig. 7 contains

sketches showing the general characteristics of the four types. A description follows:

Type 1.—Simple Hinged Chute.—This is the simplest type, and is installed on the majority of piers having a number of chutes along the side. The chute is of metal, or of timber with a metal lining. It is hinged to the coal pocket outlet, may be swung to any desired vertical angle about the fixed horizontal hinge, and in some cases has been designed with a pivot arrangement to allow a lateral swing also. The coal drops freely from the outer or lower end of the chute into the hold of the boat. The dimensions of these chutes are dependent to a large extent on the general size of the pier and the class of vessels coaled, and are largely based on the judgment of the different designers, but the most common dimensions are approximately as follows: width at hinge 5 ft. 6 in., width at lower or free end 4 ft. 6 in., depth at hinge 18 in., depth at lower or free end 10 in., the length varies from 12 to 30 ft. On several piers chutes of this type are counterbalanced by chains or cables attached to hanging weights, the counterweight chain or cable being operated over a differential drum fastened to the drum shaft of the operating winch. Chutes of this type are operated by hand winches, and in general, when not counterweighted, it requires the services of one man for about 5 min. to lower and adjust a chute from its full vertical position to the position ready to discharge coal into a vessel, and it takes four men about 5 or 6 min. to raise a chute to its full vertical position. When the chutes are counterbalanced or over-counterweighted, these operations may be accomplished by a less force and in less time. The general design indicated by Type 1, Fig. 7, is typical, but there are a number of modifications for handling and counterbalancing, all more or less effective, some of which are patented.

Type 2.—Hinged Chute Adjustable at Upper End.—This chute is a simple apron essentially as described for Type 1, but its upper end is hinged to a metal frame which slides through a vertical guide attached to the frame of a fixed vertical leg. The chute receives the coal directly from the vertical leg, which is fed from the pockets. The vertical leg is of metal, or wood with metal lining, and has several regularly spaced openings for discharge into the chute. Each of these openings is provided with a hinged plate which can be swung into a vertical position to close the opening in the side of the vertical chute, or down into an inclined position to form a floor for the vertical leg

at the opening from which it is desired to discharge. This type allows a vertical adjustment of the apron, so that coal can be delivered to the hatches of boats without too great a drop from the outer end of the apron. It is known as the Henkel patent. The vertical adjustment of the upper end, and the adjustment of swinging the apron through its vertical angle are both made by hand winches. This chute is sometimes counterweighted for both the upper and lower ends, in a manner similar to that for Type 1. The power necessary to handle and adjust this type is considerably more than for Type 1, as it requires two men about 5 or 6 min. to lower the upper end of the chute from the highest to the lowest pocket outlets, and four or five men from 12 to 20 min. to reverse the movement. In addition to this, it takes about the same number of men and the same time to swing the apron through its vertical arc, as described for Type 1. Type 2, Fig. 7, illustrates this chute. It is used on the piers of the Philadelphia and Reading Railroad at Port Richmond in Philadelphia Harbor.

Type 3.—Hinged Chute with Telescopic Leg at Upper End.—This chute is a simple apron, as described for Type 1, but its upper end is hinged to a vertical telescopic leg, which receives the coal directly from the pocket and discharges into the inclined chute. The front or slip side of the vertical leg is open, and a movable hopper, to which the apron is hinged, can be adjusted vertically throughout the length of the leg. A series of sliding plates, resting on the top of the movable hopper and sliding in guides in the front face of the leg, closes the front face of the vertical leg as the hopper is lowered. The leg being adjustable as to vertical height, and the inclined chute being hinged thereto, makes it more flexible than either Types 1 or 2, and though the coal falls freely from the outer end of the inclined chute, the height of the fall can be reduced or increased by adjusting the vertical leg. These chutes are operated by hand winches, and the time and number of men required to lower and raise the inclined chute are the same as for Type 1; and, in addition, it requires one to two men from 5 to 10 min. to adjust the movable hopper from the highest to the lowest positions, and from four to five men from 10 to 30 min. to reverse this latter operation. These chutes are generally all of metal, and in most cases both the inclined and vertical chutes are counterweighted. This type is patented and generally known as the Link Belt Chute. It is illustrated by Type 3, Fig. 7. They are used on

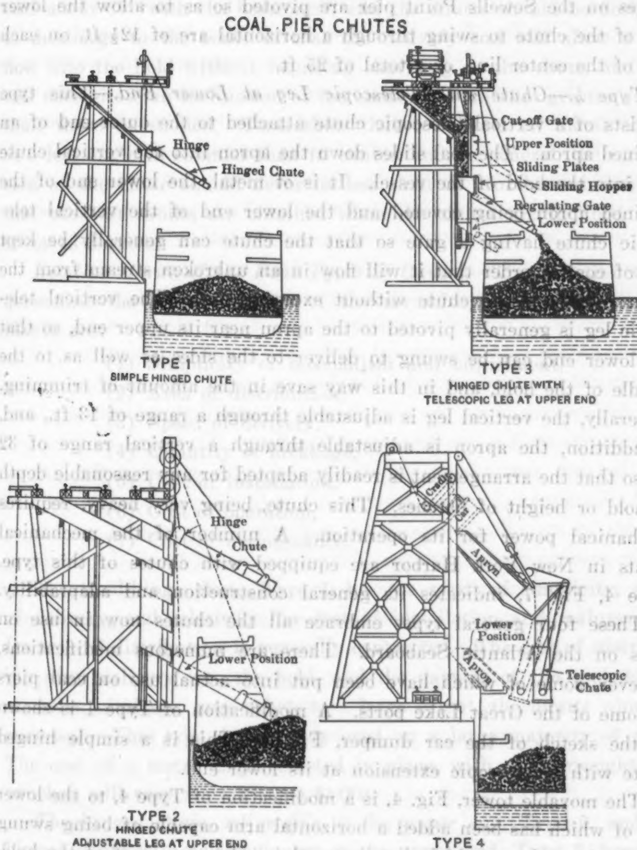

COAL PIER CHUTES
Hinge
Hinged Chute
TYPE 1
SIMPLE HINGED CHUTE
Cut-off Gate
Upper Position
Sliding Plates
Sliding Hopper
Regulating Gate
Lower Position
TYPE 3
HINGED CHUTE WITH
TELESCOPIC LEG AT UPPER END
Hinge
Chute
Lower Position
TYPE 2
HINGED CHUTE
ADJUSTABLE LEG AT UPPER END
Cradle
Apron
Lower Position
Apron
Telescopic Chute
TYPE 4
CHUTE WITH
TELESCOPIC LEG AT LOWER END

FIG. 7.

Pier No. 6, Pennsylvania Railroad, at Greenwich, Philadelphia Harbor, Curtis Bay pier and Port Covington pier, in Baltimore, and the Virginian Railway pier at Sewells Point, in Norfolk. The inclined chutes on the Sewells Point pier are pivoted so as to allow the lower end of the chute to swing through a horizontal arc of 12½ ft. on each side of the center line, or a total of 25 ft.

Type 4.—Chute with Telescopic Leg at Lower End.—This type consists of a vertical telescopic chute attached to the outer end of an inclined apron. The coal slides down the apron into the vertical chute and into the hold of the vessel. It is of metal, the lower end of the inclined apron being covered and the lower end of the vertical telescopic chute having a gate so that the chute can generally be kept full of coal in order that it will flow in an unbroken stream from the lower mouth of the chute without excessive fall. The vertical telescopic leg is generally pivoted to the apron near its upper end, so that the lower end can be swung to deliver to the sides as well as to the middle of the hold, and in this way save in the amount of trimming. Generally, the vertical leg is adjustable through a range of 13 ft., and, in addition, the apron is adjustable through a vertical range of 32 ft., so that the arrangement is readily adapted for any reasonable depth of hold or height of hatches. This chute, being very heavy, requires mechanical power for its operation. A number of the mechanical plants in New York Harbor are equipped with chutes of this type. Type 4, Fig. 7, indicates its general construction and adaptability.

These four general types embrace all the chutes now in use on piers on the Atlantic Seaboard. There are numerous modifications, however, some of which have been put into actual use on coal piers in some of the Great Lake ports. A modification of Type 1 is shown on the sketch of the car dumper, Fig. 4. This is a simple hinged chute with a telescopic extension at its lower end.

The movable tower, Fig. 4, is a modification of Type 4, to the lower end of which has been added a horizontal arm capable of being swung in a horizontal arc, which distributes the coal to any part of the hold. The device is known as an automatic trimmer, but, as far as the writer can ascertain, it has not yet been used.

In general, the height of the pier and the design of the chutes are, to a great extent, the limiting factors for any one pier, because, on the height of the delivery end of the chute depends the size of the

boats that can be coaled at that pier, without excessive costs for handling the coal. It is usually recognized that the angle of repose for anthracite coal is about 27° with the horizontal and for bituminous coal about 35° with the horizontal, and the chutes should be designed to discharge to the boats so that the delivery end will allow a free flow into the hold without requiring shoveling in the chute, and this will necessitate an inclination of the chutes of somewhat greater angles than those mentioned. Tests show that coal flows on metal chutes at a less angle in summer than in winter, and, in general, the inclination of the chute should never be less than 35° for bituminous, and that the best results are obtained when the chutes are inclined from 40 to 45 degrees.

Relative Merits of the Four Types of Chutes.—The qualities which determine the relative merits of the several types used on gravity plants are:

(1) Simplicity of mechanism and adjustment,
(2) Area of distribution,
(3) Speed of delivery,
(4) Quantity of breakage,
(5) Cost of installation,
(6) Cost of operation,
(7) Cost of maintenance.

The fixed-hinge chute, Type 1, has the merit of simplicity, speed of delivery, and relatively low cost of installation and maintenance. Its area of distribution is generally contracted. The cost of distributing after delivery into the hold, and the damage of the coal due to excessive breakage, may offset, to a large extent, the merits which it possesses. This type, however, is used on a large majority of piers. The cost of a metal chute, erected in place, with counterweight and winches, will vary from $800 to $1 000.

The hinged chute, adjustable at the upper end, Type 2, and the hinged chute with telescopic leg at the upper end, Type 3, have the merit of distributing delivery and small breakage, but the greater cost of installation, operation, and maintenance may offset their merits, to some extent, depending on the commercial conditions under which the coal is handled. The complete cost of this type, connected to the vertical legs on the pier, varies from $1 500 to $1 600.

A number of devices have been patented and tried on the ends of fixed chutes, with an idea of reducing the breakage of coal, giving a greater distribution for gravity piers, but most of them have been discarded because they were cumbersome and slow, or wore out quickly; in cases where they have been meritorious they have been discarded on account of lack of interest by the pier management, which may have felt that the quick release of vessels, a large daily tonnage, and consequent reduction in pier operating expenses, will be more important to the railroad interests than the loss in the value of the coal to the customers due to the breakage. The question is really a commercial one for each pier, as to whether the railroad's interests are or are not best served by the increased expense of installing and maintaining chutes which deliver the coal with the least breakage.

The chute with the telescopic leg at the lower end, Type 4, because of its weight, is adaptable mainly to mechanical plants or on gravity piers equipped with mechanical power for handling the chutes. This type has the merits of wide distribution, speed, and least breakage; but, for gravity piers, has a high cost of installation, operation, and maintenance, and is cumbersome in handling. The merit of distributing the coal over a considerable area, thereby reducing the cost of hand trimming is a commercial factor to be weighed against the cost of installation and of maintenance and power for operation.

On account of the varieties of bituminous coal handled from piers, different arrangements have to be provided in the various ports. In Norfolk Harbor the coal handled is the Pocahontas variety, nominally hard and lumpy, with little dust. It may be dropped from greater heights than is permissible for other varieties, and will not be badly broken. On account of breakage, particular care has to be taken in ports where soft grades of coal are handled, especially for foreign shipments.

Track Scales.—Track scales are provided wherever the business requires that the coal be weighed. They are placed at a practical location on the delivery track on the ground or pier, or at the entrance of the load yard. Empty cars are seldom weighed, but, when necessary, this may be done on the general yard scales. There is only one pier where a scale was installed on the empty return track to weigh empty cars, and it has been abandoned.

In most cases the scales are located so as to require the least shift-

ing of the cars, and be convenient for observation and the concentration of the clerical forces. Their lengths are such as to meet the requirements of the standard length of cars used, the running speed while being weighed, and whether cars are weighed while coupled or while moving singly. The standards in use by the various roads vary from 32 to 60 ft. for scales not fitted with automatic weighing devices, and from 34 to 46 ft. for those fitted with such apparatus. On the two new piers now in course of construction in Norfolk Harbor, scales 68 ft. long, with automatic weighing devices, are contemplated. The weighing capacities of the scales on the piers and in the yards in the various ports vary from 160 000 to 300 000 lb. About one-half of the scales investigated were equipped with automatic weighing devices.

Scales must have unyielding foundations, ample strength, and capacity in excess of the ordinary requirements, in order to withstand the constant use and vibrations without having their accuracy destroyed.

Power Incline Mechanism.—The equipment for a power incline usually consists of a couple of boilers, an engine, and a cable running over pulleys and drums generally attached to what is called a mule, which is arranged so as to push the car up the incline to the deck. In one instance—the pier of the Virginian Railway Company at Sewells Point, Norfolk Harbor—electrical power is used instead of steam. Some of the power inclines are not equipped with a mule, but a cable is attached directly to the front end of the car, which is pulled up instead of being pushed. The mule arrangement is considered safer for operating purposes, and requires less inspection of the draw-bars and car mechanism than the cable haul. The complete mechanical equipment, installed, will cost about \$17 000 for a 16% incline, or \$25 000 for a 25% incline.

Coal-Thawing Plants.—Three types of plant for thawing frozen coal are in use. One is simply the thawing of coal in cars while on the pier deck or in the yard by steam jets inserted in the coal, steam being furnished from a locomotive or from independent boilers. In the second type the coal is also thawed by jets of steam, but the plant is installed permanently in sheds. The third type is somewhat elaborate, the coal being thawed under cover by hot air.

As thawing is required only during severe weather, and then possibly only for short seasons, it is, at a number of piers, not considered

of sufficient importance to justify the expense of installation, and in such places frozen coal is broken by hand.

There are nine plants of the first type, located at Elizabeth, Hoboken, Weehawken, and Guttenberg, in New York Harbor, Greenwich port, in Philadelphia, Port Covington, in Baltimore, and Lamberts Point, Sewells Point, and Newport News, in Norfolk Harbor. There is one of the second type at Edgewater, and one of the third type at South Amboy, in New York Harbor.

Construction.

The construction of most gravity piers is of a temporary character, and consists of pile foundations and framed timber superstructure. Judging from the fact that a large number of these piers do not at present meet the increased requirements, their temporary character of construction was apparently justified. Some of the more modern ones, however, such as those at Sewells Point and Lamberts Point (Piers Nos. 2 and 3), Norfolk Harbor, are constructed in a permanent manner over the length of the dock, but such construction was practically made necessary because of their great height and large capacity. The piers under construction by the Norfolk and Western at Lamberts Point, and the Chesapeake and Ohio at Newport News will also be of permanent construction.

As the cargoes, operating conditions, and possible future developments are at present better understood, the modern pier, in order to meet existing conditions and anticipated developments, should be built in a more permanent manner than heretofore, so as to secure the necessary stability under the more severe operating conditions and as a precaution against danger of destruction by fire.

Operation.

The direct cost of operating a pier includes clerical force, policing, engine service, expenses for running the machinery, and the labor of dumping cars and trimming coal. The items of interest on investment, maintenance of plant, and depreciation are also proper charges which affect materially the cost of handling coal, and though not herein considered part of the operating expenses, they are fully accounted for under the heading, "Relative Merits of Different Classes and Types." The items of expense due to general office, supervision, yard, and tugboat service have not been considered in this paper, as they have no important bearing on its object.

Clerical Force and Policing.—The expenses of office force and policing vary somewhat with the capacities of the plants. On one gravity plant investigated these charges in 1911 amounted to \$8 068 for clerical force and \$1 416 for policing, a total of \$9 484 for an output of 2 078 321 tons, making an average cost per ton of about 0.46 cent. During that year the pier worked to about 90% of its estimated average capacity, or at the average rating of 180 cars per day for 270 days. If it had worked to its full average capacity of 180 cars for 300 days, the cost per ton would have been about 0.42 cent, and if it had worked to a capacity of 200 cars per day for 300 days the cost per ton would have been 0.38 cent. This cost of 0.38 cent per ton, therefore, may be considered a reasonable charge for clerical force and policing on a pier working to a normal capacity of 200 cars per day. When working to 75% of the normal capacity, the cost would be about 0.50 cent per ton.

Engine Service.—A locomotive incline type of pier working to an average capacity of 200 cars per day requires the constant use of one locomotive for pushing the cars from the yard to the pier deck. The average charge for an engine which will do the work effectively will approximate \$43 per day, the cost being made up of the following items:

Repairs	\$10.94	
Miscellaneous expenses	1.64	
Interest on cost, \$13 500 at 5%	1.85	
Depreciation " " "	1.85	\$16.28
Coal, 8 tons at \$1.00	\$8.00	
Lubrication	0.24	
Water	0.77	
Other supplies	0.31	
Engine-house expense	0.70	10.02
One engineer	\$4.00	
One fireman	2.50	
One conductor	3.60	
Two brakesmen at \$3.30 each	6.60	16.70
Total daily charge against operation		\$43.00

This engine service would amount to $12 900 per year of 300 working days, or about 0.51 cent per ton, on the basis of an average car load of 42 tons. For piers of the barney incline type, or mechanical type, where the yard is arranged so as to permit the loaded cars to drift by gravity from the yard to the barney pits, no engine service will be required, but, when not thus arranged, one engine will be required for bringing the loaded cars to the barney, at a cost of 0.51 cent per ton.

Expenses of Operating Machinery.—On the power incline type of pier, with a 22% grade, and developing a capacity of 200 cars per day, the daily charges for operating the haulage system (not including dumping cars or engine service) will be about as follows:

Coal, 6 tons at $2.50............	$15.00	
Oil, water, light, etc............	3.25	$18.25
One engineer..........................	$4.00	
Two firemen at $2.40..........	4.80	
Two signal men " 2.00..........	4.00	
Four car riders " 2.00..........	8.00	
One general man " 2.40..........	2.40	23.20
Total daily charge..................		$41.45

This is equal to a cost of 0.49 cent per ton. If the plant should work to only 75% of its average daily capacity, the cost per ton would be about 0.60 cent, assuming that only the coal consumption is reduced.

On a modern mechanical car dumper, operated by steam, with an average capacity of 200 cars per day, the daily cost for operating the machinery, including dumping cars, but exclusive of the engine service, will be about as follows:

Coal, 15 tons at $2.50...........	$37.50	
Oil, water, incidentals, etc........	8.00	$45.50
One engineer....................	$4.00	
Two firemen at $2.40...........	4.80	
One cradle and apron operator....	3.00	
One car handling machinery.....	3.00	
One chute operator...............	2.50	
Four car riders at $2.00.........	8.00	25.30
Total daily charge..............		$70.80

This is equal to a cost of 0.84 cent per ton. If the plant should work to only 75% of its average daily capacity, the cost per ton would be about 0.98 cent, assuming that only the coal consumption is reduced.

Labor of Unloading Coal.—On gravity piers, with a maximum capacity of 300 cars per day and an average of about 200 cars per day, it requires about eight gangs of eight men each on the deck, and one general foreman, for unloading 8 cars at a time into boats in two slips. It takes one gang about 2 min. to dump a car of coal, under the most favorable conditions, and as many as 45 min. when the coal is frozen. In general, 3 cars per chute per hour is considered good work.

On some piers the unloading of the coal after the cars are placed on the deck is done by agreement with the men on the basis of a unit price per car. At a pier in New York Harbor, average capacity 150 cars per day, a car of any size is unloaded for $1, or at about 2.38 cents per ton. At a pier in Philadelphia Harbor, average capacity 100 cars per day, cars are unloaded at the following rates:

60 000-lb. wooden gondolas, $1.00 = 3.73 cents per gross ton.
100 000 " self-dumping cars, 1.50 = 3.36 " " " "
115 000 " steel gondolas, 2.00 = 3.88 " " " "

On a pier in Baltimore, average capacity 180 cars per day during 1911, the cost for labor alone in dumping cars was 1.89 cents per ton. This may also be considered a reasonable price for a pier of the locomotive incline type, working up to a normal capacity of 200 cars per day, and the cost will be the same for a pier of the power incline type. For mechanical plants which dump the cars bodily, the cost of unloading is included in the cost of handling the plant.

Trimming Coal.—Trimming is usually paid for by the hour, and the cost will depend largely on the ruling wages, the type of chutes on the pier, and the character of the vessels loaded. For simple hinged chutes, which practically do no distributing, there is necessarily more trimming than for those which are telescopic and adjustable. The cost of trimming for one large plant in Baltimore Harbor during 1911 was 3.42 cents per ton. In general, where trimming is done under agreement with stevedore companies, the price per ton is about:

3 cents for open-top boats, such as barges and lighters,
5 " " single-deck boats,
7 " " large double-deck colliers.

In mechanical plants with the ordinary car dumper, and telescopic chute at the lower end, the amount of hand trimming is estimated at about 90% of that required when boats are loaded from gravity plants. Mechanical trimmers attached to the lower end of the telescopic leg would probably require only 75% of the amount.

For estimating purposes, in connection with 200-car average daily capacity piers, when coaling to all classes of boats, the following costs per ton for trimming will represent fairly well the average prices:

(a) Gravity piers, chutes having adjustable legs...3.42 cents.
(b) Car-dumping machines with telescopic chute..3.08 "
(c) Car-dumping machine with mechanical trimmer.2.56 "

Summary.—Reasonable and fair charges per ton for the operating expenses of locomotive incline type, power incline type, and car-dumping type of plant, each working to its normal capacity of about 200 cars per day, and when yard grades are such as to permit the gravity delivery of cars from the load yard to the barney pits without engine service, are given in Table 10.

TABLE 10.—CHARGE PER TON FOR OPERATING EXPENSES FOR VARIOUS PLANTS; FULL NORMAL CAPACITY.

Items.	Locomotive incline.	Power incline.	Car-dumping machine.
	Cents.	Cents.	Cents.
Office and policing	0.38	0.38	0.38
Engine service	0.51		
Labor and fuel for machinery	...	0.49	0.84
Unloading cars	1.89	1.89	....
Total, exclusive of trimming	2.78	2.76	1.22
Trimming coal	3.42	3.42	3.08
Total, including trimming	6.20	6.18	4.30

In case the piers should work only to about 75% of their normal capacities, which is nearer actual practice, the operating charges per ton, other conditions being the same as above, will be as given in Table 11.

In cases where engine service is required for bringing the loads from the yards to the barney of barney-incline and car-dumping plants, the cost of such service should be added to that of operating either the barney-incline or car-dumping types, and, under such conditions, the operation cost per ton for the three types would be as given in Table 12.

TABLE 11.—CHARGE PER TON FOR EXPENSES WHEN OPERATING TO 75 PER CENT. OF NORMAL CAPACITY.

Items.	Locomotive incline.	Power incline.	Car-dumping machine.
	Cents.	Cents.	Cents.
Office and policing	0.50	0.50	0.50
Engine service	0.51	....	....
Labor and fuel for machinery	....	0.60	0.98
Unloading cars	1.89	1.89	....
Total, exclusive of trimming	2.90	2.99	1.48
Trimming coal	3.42	4.32	3.08
Total, including trimming	6.32	6.41	4.56

TABLE 12.—OPERATING COSTS.

	Locomotive incline.	Power incline.	Car-dumping machine.
A.—PLANTS WORKING TO FULL NORMAL CAPACITY.			
	Cents.	Cents.	Cents.
Operation, exclusive of trimming	2.78	3.27	1.73
Total, including trimming	6.20	6.69	4.81
B.—PLANTS WORKING TO 75 PER CENT. OF NORMAL CAPACITY.			
Operation, exclusive of trimming	2.90	3.50	1.99
Total, including trimming	6.32	6.92	5.07

The charges for interest, maintenance, and depreciation also enter into the cost per ton for handling coal, and are considered in connection with and under the heading "Relative Merits of Different Classes and Types." Therefore, the operating costs alone, as contained in the foregoing tabulations, should not be considered as representing the relative economy of the different plants.

RELATIVE MERITS OF DIFFERENT CLASSES AND TYPES.

Each type of pier herein described has certain features which may give it a preponderance of merit for that particular location and those special service requirements for which it is best adapted. Therefore, the relative merits of the different types should not be judged independently of a consideration of the particular local situation and requirements which the construction of the plant will have to meet. In every case, however, where the local conditions will permit of the adoption of a gravity, mechanical, or combination type, the rela-

tive merits should be measured by the relative efficiencies for the location considered. For a proper determination of these efficiencies the following items must be examined:

(1) Discharging capacities,
(2) Ability to make simultaneous deliveries to several vessels,
(3) Relative freedom from possible breakdown which will seriously delay operation,
(4) Relative breakage of coal during delivery,
(5) Cost of plant,
(6) Interest on cost,
(7) Depreciation of plant to be taken care of by an annual fund drawing compound interest, which will amount to a sufficient sum to renew the various parts of the plant at the end of their estimated serviceability period,
(8) Cost of operation, which includes cost of handling and dumping cars, policing, office expenses, engine service, expenses for operating machinery, trimming, etc.

Items 6, 7, and 8, representing interest, depreciation, and operation, are the annual charges in connection with the plant, and, when stated in terms of cost per ton of coal output, the relative cost per ton for the different types will represent their relative economy. The plant which may have the merit of greatest economy in cost per ton of coal output, may be deficient to such an extent in regard to capacity, simultaneous delivery to several vessels, possible breakdown, and coal breakage, as to offset its apparent economy; therefore, all the eight items mentioned, and not merely apparent economy, should be fully considered when determining the relative merits of different types for a particular location.

Relative Economy.

For the purpose of determining the relative economy of a locomotive incline of Type A6, a power incline of Type A8, and a mechanical car dumper of Type B2, detailed estimates have been prepared covering:

(1) Temporary construction, namely, pile and timber work as far as practicable, that exposed in water being treated.
(2) Partly permanent construction, namely, trestle approaches, bulkheads, barney inclines, and empty return trestle for me-

chanical plants, of pile and timber construction as far as practicable; and the pier substructure and superstructure, from the bulkhead to the sea end, with the exception of barney and return trestles for mechanical plants, of steel and concrete as far as practicable.

(3) Permanent construction, namely, all parts, including approaches, to be of steel and concrete as far as practicable.

These estimates include everything except right of way, grading, and tracks, and are based on the following assumptions:

(*1*) *Local Conditions.*—Local conditions suitable for the construction of an economical design of each type, the grade in the load yards being such as to give gravity delivery of loads to the foot of the barney incline for Types A8 and B2 plants, thereby not requiring engine service for this purpose.

(*2*) *Normal Average Capacity.*—The normal average capacity of each plant to be 200 cars per day for 300 days per year, or an annual output of 2 520 000 gross tons, based on the average of 42 tons per car.

(*3*) *Dimensions.*—For locomotive and power incline types: length from bulkhead, 500 ft.; height above mean tide, 60 ft.; width at lower deck, 70 ft.; two slips, each 500 ft. long, 200 ft. wide, and 30 ft. deep.

For mechanical car dumper: pier length, 800 ft.; width, 55 ft.; only one slip, 800 ft. wide and 30 ft. deep.

(*4*) *Tracks and Grades.*—Locomotive incline: one track, 3% grade; pier deck, 2 tracks, 1% grade; empty return, one track, about 2% grade.

Power incline, Type A8: one track, 20% grade; pier deck and empty return, same as for locomotive incline.

Mechanical car dumper: barney incline; one track on 12% grade, and empty return over track on grade varying from 2 to 1 per cent.

(*5*) *Cost of Plant.*—Present average ruling prices.

(*6*) *Interest on Investment.*—5% annually.

(*7*) *Depreciation.*—The assumption is made that at the end of 25 years the conditions will have changed to such an extent as to necessitate the re-design and construction of all temporary work and machinery, and at the end of 50 years all permanent work, except dredging in slips. The charges for this depreciation are taken care of by an annual renewal fund drawing compound interest at 4 per cent.

(8) *Maintenance Charges per Annum.*—

Temporary work.—	Pier substructure and bulkheads..	5%
	Pile foundations in approaches...	5%
	Untreated framed timberwork...	10%
Permanent work.—	Steelwork	2%
	All other construction...........	1%
Machinery.—	Power incline, car dumper, scales, chutes, etc.................	10%
Dredging.—	....	1%
Operation.—	From Table 12.	

The results of the estimates are summarized in Table 13, which shows the relative economy of the different types and kinds of construction, in terms of the total cost per ton of coal output, and when based on the assumed conditions.

It will be observed that, under the assumed conditions:

(a) The mechanical car-dumper type will not only be the least expensive in first cost, but is also by far the most economical in total charges per ton of coal output;

(b) The power incline type is somewhat less expensive in first cost than the locomotive incline, and is also a little more economical in total cost per ton of coal output;

(c) That though the permanent construction, for any of the three types, will necessarily cost more than temporary or partly temporary work, it shows more economy in the total cost per ton of coal output.

For plants of greater output and greater heights than assumed, the cost per ton of coal will vary from that given in Table 13, but the relative economy of the different types will not be affected materially. If the annual output should be about 75% of that assumed, or, in other words, if the piers should work only 225 days per year instead of 300, as assumed, then the car dumper would still remain the most economical in first cost and cost per ton of coal output, but the locomotive incline, though costing a little more to construct than the power incline, would be more economical than the latter in total cost per ton of coal output.

In the foregoing comparison, although the mechanical car-dumper type, B2, is the most economical of the three types considered, under

the conditions assumed, yet it must be remembered that a single car-dumping machine can deliver coal to only one vessel at a time, and, where conditions require simultaneous delivery to several vessels, it becomes necessary to construct at least two machines, in which case economy no longer exists.

TABLE 13.—Comparative Estimates of Cost.

Type of pier.	Cost of plant.	Cost per Ton of Coal, in Cents.						
		Int.	Depr	Maint	Operation.			Total charges per ton.
					From deck.	Trim.	Total.	
Temporary Construction.								
Loco. incline, A6	$807 720	0.61	0.25	0.76	2.78	3.42	6.20	7.82
Power incline, A8	299 280	0.59	0.24	0.74	2.76	3.42	6.18	7.75
Car dumper, B2	279 640	0.55	0.22	0.66	1.22	3.08	4.30	5.73
Partly Permanent Construction.								
Loco. incline, A6	373 790	0.74	0.18	0.56	2.78	3.42	6.20	7.68
Power incline, A8	365 350	0.72	0.17	0.54	2.76	3.42	6.18	7.61
Car dumper, B2	334 640	0.66	0.16	0.52	1.22	3.08	4.30	5.64
Permanent Construction.								
Loco. incline, A6	431 400	0.86	0.12	0.37	2.78	2.42	6.20	7.55
Power incline, A8	395 980	0.79	0.13	0.41	2.76	3.42	6.18	7.51
Car dumper, B2	340 260	0.67	0.14	0.47	1.22	3.08	4.30	5.58

A plant similar to Type B4, Fig. 4, which is composed of a fixed car-dumping machine and a movable-tower car-dumping machine would be more expensive in first cost than the car dumper, Type B2, but would cost somewhat less than either the locomotive or power incline of permanent construction. The total cost per ton of coal output would be about 0.5 or 0.6 cent less than for the locomotive or power incline type, about 1.4 cents more than for the simple car dumper, B2, on the assumption that the chute is not equipped with a mechanical trimmer. Type B4 coals vessels from the tower dumper, which can be moved along the dock to the hatches of a large vessel, or to a number of barges moored to the pier, and the tower can be moved more speedily than the vessels and barges. It will require a pier length of about 500 ft., as compared with 800 ft. for the dumper alone.

Comparison of Combination with Other Classes.

Combinations of car-dumping machines with special cars and elevators or inclined planes for raising the special cars to the upper decks of the piers, such as the proposed Norfolk and Western Pier No. 4 at

Lamberts Point, the proposed Chesapeake and Ohio Pier No. 9 at Newport News, and the existing Virginian Railway pier at Sewells Point, are constructed for the purpose of meeting requirements for which the ordinary gravity types and the mechanical dumper alone are not so well adapted, and, therefore, they form a class by themselves for special work. These, of course, can be constructed in locations which also favor the ordinary gravity types or the simple car-dumping machine, but, under such conditions, there is no apparent economy in constructing a combination plant composed of a dumping machine and a gravity pier, when either one alone would serve the purpose.

Selection of a Type of Pier.

The selection of a type of gravity pier should not be made until the relative merits for the local conditions have been fully determined in the manner indicated in the preceding discussion. It is not merely a matter of adopting the plans of some pier which is in operation elsewhere, as the tendency too often prevails at present, but is a matter which involves a most thorough consideration of the dimensions and cargoes of the vessels to be loaded, the required maximum daily capacity, the probable future developments, the local conditions which affect approaches, grades, track layout, etc., and finally the preparation of a design which will meet these conditions economically.

The selection of a type of mechanical plant is also, as in the case of a gravity plant, not merely a matter of fancy or adoption of a type already built, but one of efficiency for conditions to be met, and requires the same careful study of controlling conditions and economy. As a rule, mechanical plants heretofore have not been installed where conditions are equally suitable for the construction of a gravity plant. There is a feeling, more or less unfounded, against too much machinery, and this feeling prejudices operators against such types; nevertheless, their utility is strongly indicated, and their economy is beyond doubt in certain locations, regardless of the question of individual feelings.

As indicated in the previous discussion, the combination of mechanical car dumpers with gravity piers, such as those in use and under construction in Norfolk Harbor, comprises a special class of plant adapted for service conditions which render the construction of an ordinary gravity plant or car-dumping machine impracticable.

The selection of a type evidently rests on the proper determination of its relative merits in meeting the following conditions:

(1) Free delivery of coal to all sizes and classes of boats which take coal or are likely to take coal in the port under consideration.

(2) The delivery of coal without breakage to such an extent as to be objectionable to the users,

(3) The simultaneous delivery to several vessels on one or both sides if required,

(4) Hourly capacity such as to give prompt release to vessels coaling,

(5) Economical cost per ton of coal output.

It has been shown that the car-dumping machine, Type B2, is the most economical in cost per ton of coal output, but can coal only one vessel at a time. The vessel, if a large one, must be moved so as to bring its hatches within the range of the chute, and must be taken away from the pier before another vessel can coal. These disadvantages should be weighed against its economy, and, where local conditions require the simultaneous loading of several vessels, this type is not suitable unless more than one machine is installed, and this can be done only at the sacrifice of considerable economy.

It has also been stated that the mechanical plant, Type B4, composed of a fixed car dumper and a movable-tower car dumper, though not as economical as the fixed car dumper alone, is more economical in cost per ton of coal output than either the locomotive or power incline types. It has the advantage of coaling simultaneously to two boats, one of which may be a large collier, and both boats may be kept moored to the pier while being coaled. There are no plants of this type in operation, as far as the writer knows, but it is a practicable type, and worthy of consideration.

It has also been shown that, when the yard can be constructed on such grades as to feed the coal cars to the barney pits, the barney or power incline type is slightly more economical than the locomotive incline type, but, if the local conditions are such as to require a locomotive for delivering the coal cars from the yard to the barney, then this type is not as economical as the locomotive incline type, but it has

an advantage in many locations where the space back of the bulkhead is contracted.

As a general proposition, where the location will permit of its construction, and vessels coaling cars are of moderate height, and where the capacities are between 20 and 30 cars per hour, the locomotive incline type has proved more satisfactory than any of the others, on account of the simplicity of its construction and freedom from heavy or complicated machinery.

Proper Design and Construction.

After having determined which type of pier has a preponderance of merit for the local situation and the conditions under which it is to be operated, it should be designed and constructed in such a manner as to obtain the greatest possible efficiency and economy consistent with the type selected.

The best results cannot be obtained by copying the plans of some similar pier, because most piers thus far designed have defects which impair their efficiency, and though in some cases the defects of design have been remedied after the piers have been placed in service, the original plans in all probability remain unchanged.

The proper determination of heights, lengths, widths, number of tracks, grades, switchbacks, construction of bins or pockets and chutes, etc., though usually considered as details of the design, are just as essential for the effective operation of the plant as the proper design of the carrying parts is for the safety of the structure.

All the so-called details have been rather fully discussed herein, and therefore a repetition is not necessary. The salient points, however, may be briefly summarized, as follows:

(1) The height of a gravity pier above mean tide should be from 33 to 43 ft. above the highest hatches of vessels coaling at the port, and will depend on the type of chute and the storage capacity of the bins.

(2) The length from the bulkhead to the sea end of a gravity pier should be not less than about 700 ft., in order to accommodate Government colliers and the probable future steamers of the New England Coal and Coke Company.

(3) The least practicable width for gravity pier coaling from two sides is about 70 ft.

(4) Grades on locomotive inclines should preferably not exceed 3%; on barney inclines to gravity piers, 18%; on barney inclines to mechanical plants, 12%; and on empty return tracks, 2 per cent. Grades for gravity movement of loads should preferably be not less than from 0.8 to 1.25%, depending on southern or northern location. All changes in grades should have easements.

(5) Switchbacks are preferable to pivot tables, and should be constructed so that grades may be readily adjusted to meet varying conditions of weather and temperature.

(6) Tops of pockets or bins should have a length of at least 12 ft. and a width of at least 9 ft., for receiving coal from either a hopper or gondola car. The slopes should be not less than 40° from the horizontal, and preferably 45°, with leads to the chute designed so that they will not form a wedge at the pocket outlet. A pocket free from sharp corners, and constructed on lines of curves rather than of angles will permit the coal to flow more freely.

(7) Metal chutes with adjustable legs at the pier connection, or a similar type, are superior to the simple hinged chute. Where electric power is available, and the pier has a large output to cargoes of various sizes, the adjustments should be controlled by motors. For a free flow of coal, the inclination of the chute from the horizontal should be about 40° for bituminous coal. Chutes with telescopic legs at the discharging end cause less breakage of coal than other types, and require less trimming.

The question as to whether the construction shall be temporary, partly permanent, or wholly permanent is one of policy, to a large extent. In most of the piers, both substructures and superstructures are of temporary construction, and all gravity piers thus far built are of temporary construction back of the bulkhead. Temporary work is the least expensive in first cost, but is less economical than either partly permanent or wholly permanent construction, as the total cost per ton of coal output will be the highest when interest, depreciation, and maintenance are taken into consideration.

A pier should be properly designed, not only to meet requirements as to normal and maximum hourly output, but also the very probable general increase in cargoes, and the capacity and weight of coal cars. Mechanical car dumpers are now designed for handling coal cars of 100 tons capacity. The construction, therefore, no matter whether temporary or permanent, should by all means be substantial and simple in character, and, in most cases where piers will probably meet conditions for a period of 25 years, it will be true economy to construct all work from the bulkhead to the sea end, for both substructure and superstructure, in a permanent manner, even if the approach trestlework and empty return be of temporary construction.

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Paper No. 1292

MODERN PIER CONSTRUCTION IN NEW YORK HARBOR.*

By Charles W. Staniford, M. Am. Soc. C. E.

With Discussion by Messrs. E. G. Walker, Edwin J. Beugler, Harrison S. Taft, F. R. Harris, J. P. Snow, Tyrrell B. Shertzer, C. H. Stengel, L. D. Cornish, L. J. Le Conte, Chandler Davis, R. T. Betts, and Charles W. Staniford.

Introduction.

The present general port activity and agitation for a modernization and expansion of the dock and wharfage system in New York City indicate that at last the community at large seems to realize the necessity of keeping its producing plant, the harbor, up to date and at the top notch of efficiency.

It has been said that "all roads lead to New York." Never was this more true than to-day. New York in 1913 is the world's greatest seaport, its greatest factory, and its largest department store.

There can be no question that New York City's supremacy as a manufacturing and distributing center is due to wise adaptation of its magnificent harbor. The phenomenal increase in the size of vessels, necessitating longer docks, and the great and constant increase in tonnage entering the harbor, both demand determined action in port development.

New York is approximately equidistant from the ports of Northern Europe and South America. Therefore, it will undoubtedly receive additional impetus in its commerce and shipping on the completion of

* Paper presented at the meeting of September 3d, 1913.

the Panama Canal. Further, on the completion of the New York State Barge Canal, it will have a direct all-water route to the Great Lakes and the North Middle States and Canada, and the Cape Cod Canal will tap New England commerce.

Far-sighted public men foresee that New York is destined to become the world power in commerce and industry, and realize that it must prepare for the future intelligently and on broad lines, permitting gradual expansion as required by commercial necessity.

Growth and Development of the Harbor.

In this period of harbor activity, it will be of interest to both the public and the engineer to describe the gradual development of the harbor, as such development was first systematically undertaken by the City, when, in 1870, the Department of Docks was organized for this purpose, and to show the types of pier construction evolved.

In considering the history of dock development in the City of New York, through the instrumentality of the Department of Docks, it must be borne in mind that, in its early days, the Department was greatly handicapped in its progress by the fact that the City actually owned only a very small portion of its great water-front, most of it having passed, by successive water grants, into the control of private interests.

It had been the policy of the New York State Government, prior to the organization of the Department of Docks, to give to corporate interests or private persons grants of land under water in that portion of the present City outside of Manhattan, the object and hope in making such grants being that such cession of land under water would be a sufficient incentive for the investment of private capital in the development of the port.

The hopes of the State and City were fully realized; in fact, they were so generally fulfilled that when the port authority, created by the Legislature in establishing the Department of Docks for the purpose of intelligent development under municipal control, began to consider the expansion of wharfage facilities to meet the demands of the growing commerce, it found but little actual water-front in possession or under control of the City.

That the early City authorities used wisdom and foresight in their work of providing for proper expansion of the harbor is shown by the fact that, through their sagacity and good judgment, the number of

piers in the harbor, owned by the municipality of New York, grew from 107 in 1868, valued at $20 000 000, to 232 in 1913, valued at $100 000 000 or more.

There has accrued, therefore, to the City, a return on its investment in this development of the dock system, a large sum of money in increased valuation and annual rent receipts, the latter aggregating in round figures about $4 000 000 per annum, the interest at 4% on a capital of $100 000 000.

It will be seen that, at the outset, the Department of Docks, concluded that proper growth and expansion of the harbor under municipal control depended on the acquisition and control of water-front property; and since the organization of the Department of Docks, this has been the policy followed by the City.

When, in 1870, the municipal authorities undertook the burden of increasing the wharfage facilities of the harbor, and of procuring funds for this purpose, it became necessary, as a basis for their work, to determine on some economic form or type of construction, both in regard to the pier structure proper and also the general location with respect to the available shore front, whereby the maximum wharfage accommodation could be developed without excessive or prohibitory cost.

The limited funds available and the small extent of water-front lands under the actual control of the City called for the greatest economy in space, the land requiring intense development in order to obtain the greatest possible extent of wharfage.

Bond issues to be applied to the development of wharfage had to show the same return on the investment, when executed by the City authorities, as if these finances were handled by private parties or corporate interests. Therefore, what might be termed the "principle of economy in expenditure of land and funds" was, of necessity, followed, and this principle was generally adopted by private interests as well, the consequent intensive use of the water-front resulting in the adoption of a uniform method of development by a definite system, namely, parallel piers generally at right angles to the general direction of the water-front, with intervening slips wide enough to accommodate vessels of the type intended to berth at the piers.

This parallel system of economically constructed piers, with its resulting economy in space occupied and capital expended, was undoubtedly one of the greatest factors in stimulating the development

and expansion of the wharfage facilities in the harbor, and in keeping them abreast of the constant increase in shipping and commerce. It has also created by far the greatest wharfage space of any harbor in the world.

The wooden pier, consisting of a timber deck and floor system supported by timber piles, became the adopted type. It was cheap, durable, and readily adaptable to all classes of shipping. One of the most important characteristics of piers of this type is the ease and economy with which it may be removed entirely, reconstructed wholly or in part, or expanded, at a low cost, to meet the increasing needs of commerce and shipping. A dock or system of docks sufficient to accommodate the shipping at the time it was built, might be found to be inadequate and obsolete within a comparatively short term of years, a complete re-arrangement being then necessary.

With timber structures, this transformation or reconstruction is a simple, rapid, and economical undertaking; it is difficult and costly with structures of stone, concrete, steel, etc. The use of concrete piles, reinforced concrete substructures or similar forms of construction, therefore, would not only have resulted in high first cost of construction, but the difficulty and expense incidental to the periodical removal, reconstruction, or expansion of dock structures of this type, as necessitated from time to time by the growth of shipping, would have rendered harbor construction work, as a revenue-producing municipal investment, practically impossible, and, consequently, would have greatly retarded the development of the harbor.

Types of Pier Construction.

The United States Government, by virtue of its power to control all navigable waterways in the country, established along the entire waterfront or shore line of New York Harbor two lines: one the bulkhead line, which limits the extent outshore of the solid filling or reclaimed land under water; the other, the pierhead line, which determines the limit to which piers may extend beyond the bulkhead line. These piers must be of such construction that the free flow of the tidal water shall remain uninterrupted by the supporting columns. This construction, being a condition wisely insisted on by the Government to preserve tidal conditions and currents, governs, to a great extent, the handling of vessels, particularly of large ones, and affects the sanitation of the City,

PLATE VI.
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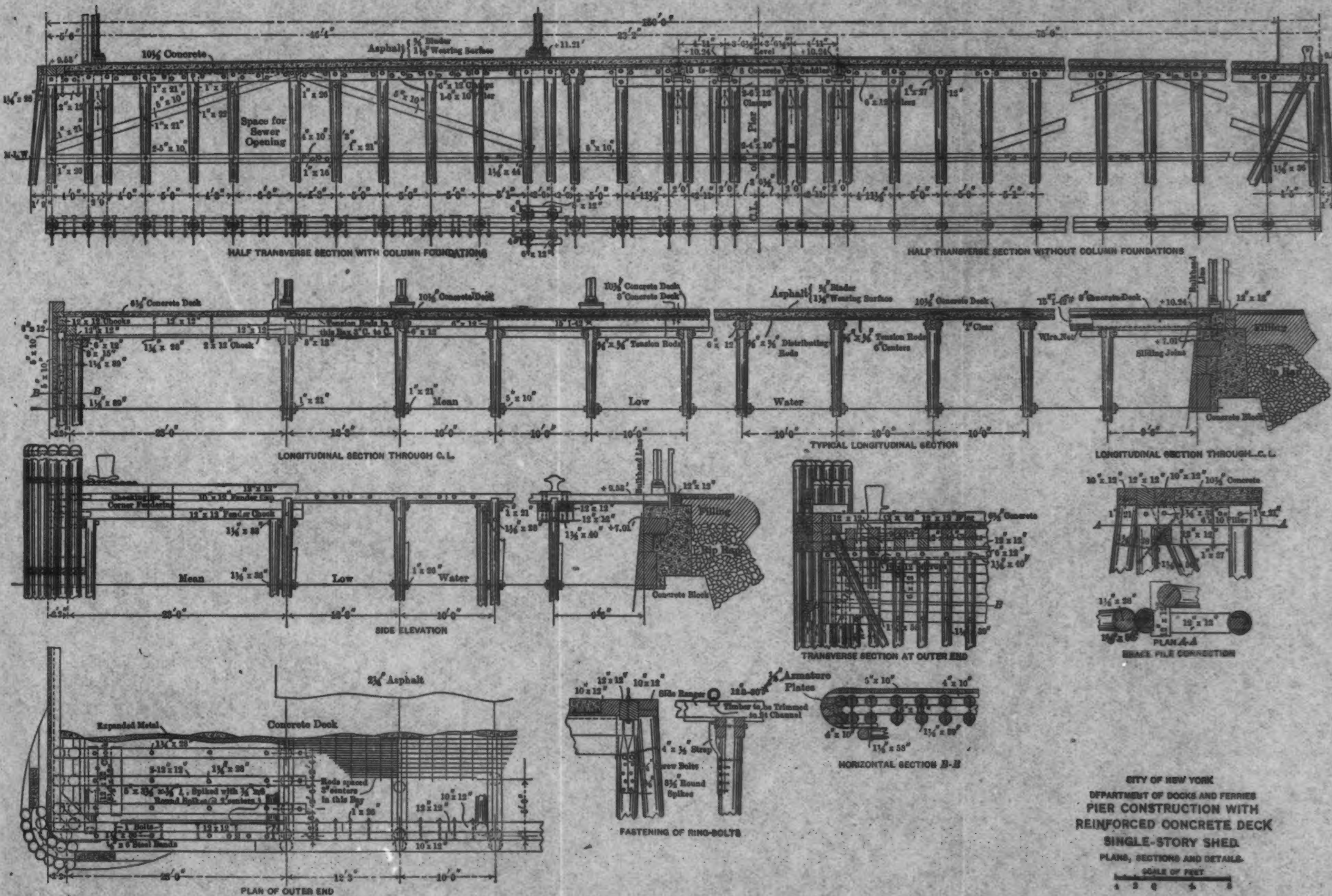

CITY OF NEW YORK
DEPARTMENT OF DOCKS AND FERRIES
PIER CONSTRUCTION WITH
REINFORCED CONCRETE DECK
SINGLE-STORY SHED.
PLANS, SECTIONS AND DETAILS.
SCALE OF FEET

in that it prevents the accumulation of sewage and refuse which would occur in closed slips. With open slips, such matter is carried away by, and disseminated in, the tidal flow. All pier construction is limited to the area included between the bulkhead and pierhead lines.

The pier which meets these requirements, and was adopted by the City in its early history as the type of structure for berthing vessels (and also adopted by all private and corporate interests), is a wooden structure throughout, consisting of a deck resting on piles driven into the mud or hard bottom. The physical features of the harbor, the geological formation of the bottom, and the condition of the water, fortunately permit the adoption of this type of construction, which, in many other parts of the world, is not adaptable because the life of the timber itself in the water would not be permanent or fairly long-lived. Wood-boring animals, the teredo, limnoria, etc., are very little in evidence, and, therefore, wooden piles are practically permanent below the water-line in almost all parts of New York Harbor.

The prominent objectionable feature to wooden pier construction is the expense necessitated by the constant repairs of the deck sheathing and the continuous wear and tear of the fender system extending along the sides and outer ends of the piers. As to the remainder of the structure, piles, floor system, etc., its maintenance and repair is very economical and consists generally in the replacement, from time to time, here and there, of decayed portions of the timber above mean low water only, at inconsiderable expense.

Until seven or eight years ago, the piers were generally built with decks of yellow pine, 4 in. thick, laid on a system of yellow pine floor structure of rangers and stringers. This deck plank in turn was covered with a second layer of either 3- or 4-in. plank sheathing, laid diagonally or at right angles to the deck proper, to form a wearing surface for the traffic.

Constant repairs and renewal of this deck sheathing, caused by the wear and tear of team traffic, is augmented in great measure by the moisture, horse urine, etc., which saturates the wood and eventually finds its way to the underlying deck and rangers. This forms the greatest item incident to the expense of pier maintenance, the average life of the sheathing for most busy piers being about 6 years, or requiring a 17% renewal annually. As the cost of the deck sheathing is generally about 12% of the total cost of a pier, it will be seen that these

sheathing repairs would aggregate 2% per annum of the cost of the entire structure.

New Pier Construction Practice.

Notwithstanding the necessity for constant repairs to the deck sheathing of the wooden pier, the parts of the remainder of the structure —rangers, caps, stringers, piles, and bracing—give excellent service. Maintenance is economical, the average life of the structure above mean low water line being from 20 to 25 years, the repairs aggregating an entire renewal above low water in that period of time. As the life of the piles supporting the structure is practically permanent when submerged below the water, the entire structure can be rebuilt after this period and made practically new by "bench capping" such piles as may be decayed above the water line and renewing the stringers, caps, deck, and sheathing; in other words, the pier structure proper, after a life of 25 years, is readily susceptible of renewal above the water line, the supporting piles below that line being to all intents and purposes permanent.

It will be readily seen that the life of the wooden pier structure would be prolonged still further, and the cost of maintenance and repairs reduced, by the elimination of the objectionable wooden deck sheathing and its replacement by some form of deck impervious to moisture and resisting the wear and tear of traffic.

It was with the object of eliminating this large repair expense incidental to the maintenance of the sheathing, and reducing maintenance cost generally, that the Engineering Bureau of the Department of Docks and Ferries, under the direction of J. A. Bensel, Past-President, Am. Soc. C. E., then Commissioner of Docks, about seven years ago, began a serious investigation and study of the problem of producing a permanent deck surface supported by timber piles, assumed as permanent below the water line.

This study has resulted in the entire elimination of the old style of wooden deck in new structures, and the production of a new type consisting of reinforced concrete laid directly on the transverse cap system of the wooden pier substructure. This concrete is laid in slabs, spanning the pile bents practically as simple beams.

This new type of deck eliminates not only the 4-in. deck sheathing, but also the 4-in. deck proper and the underlying 12 by 12-in. yellow pine ranger system longitudinally of the pier on top of the transverse cap system, further increasing the life of the substructure.

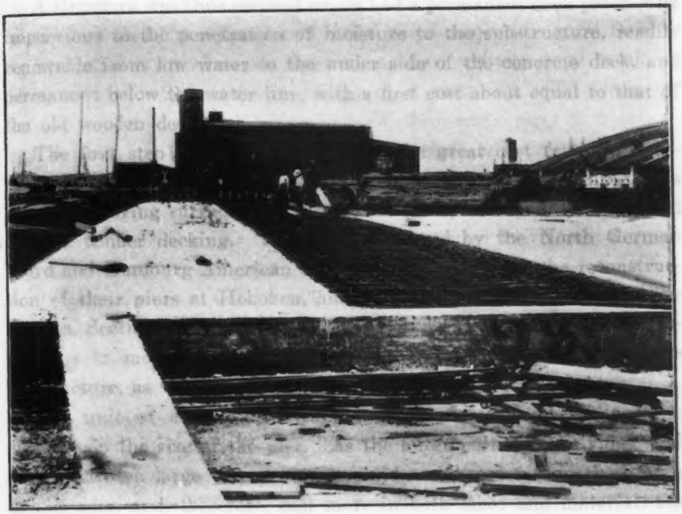

FIG. 1.—STRIP OF CONCRETE DECK AND STEEL REINFORCEMENT.

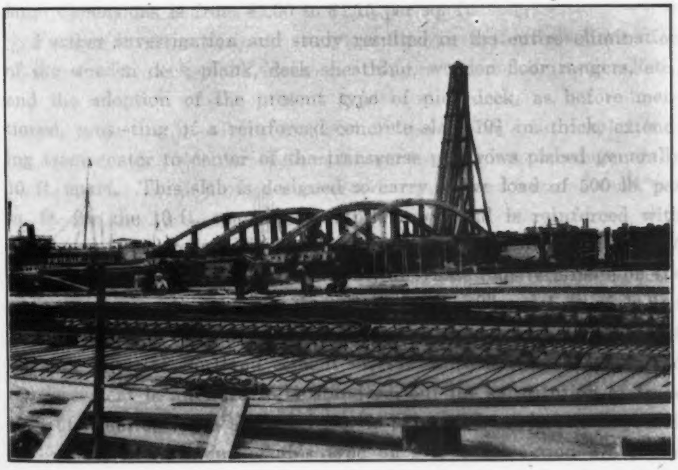

FIG. 2.—TENSION REINFORCEMENT AND TEMPLATES FOR HOLDING IN POSITION.

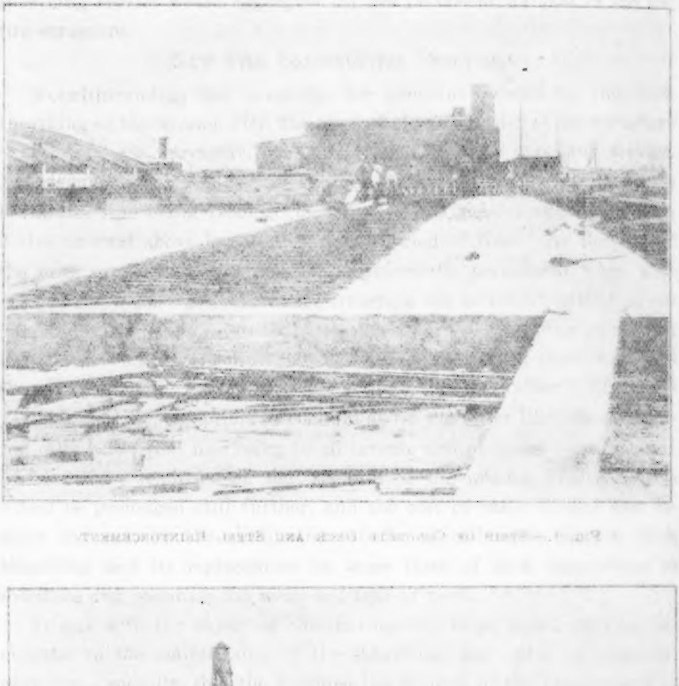

FIG. 1.—SPAN OF CONCRETE DECK AND STEEL REINFORCEMENT.

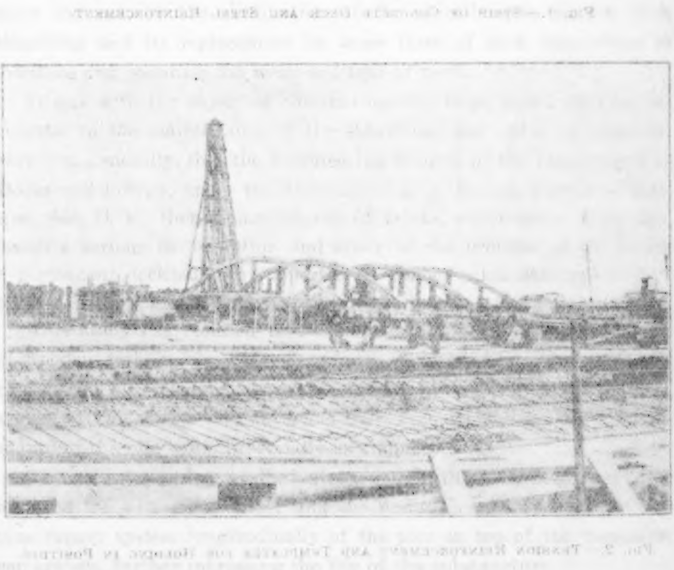

FIG. 2.—TRUSSED REINFORCEMENT AND TEMPLATES FOR HOLDING IN POSITION.

A structure was thus evolved which had a permanent deck practically impervious to the penetration of moisture to the substructure, readily renewable from low water to the under side of the concrete deck, and permanent below the water line, with a first cost about equal to that of the old wooden deck pier.

The first step in the elimination of the great cost factor, the renewal of the deck sheathing, was the replacing of this sheathing by a concrete wearing surface, from 4 to 6 in. thick, laid directly on the old type of timber decking. This type was used by the North German Lloyd and Hamburg American Steamship Companies in the reconstruction of their piers at Hoboken, and by the City of New York in the Chelsea Section piers, the result being a deck surface which is impervious to moisture and urine, and, therefore, a protection to the substructure, as well as a saving in maintenance.

The unit of cost of construction of a pier depends in a large measure on the size of the pier. As the outer portions, the sides, and outer end of a large pier are more rigid and heavier than those of a smaller pier, and, therefore, cost more in both labor and material, the relative cost per square foot of a short pier is considerably larger than that of a long one. The average cost of the old wooden deck pier of large dimensions is from $1.00 to $1.15 per sq. ft.

Further investigation and study resulted in the entire elimination of the wooden deck plank, deck sheathing, wooden floor rangers, etc., and the adoption of the present type of pier deck, as before mentioned, consisting of a reinforced concrete slab, 10½ in. thick, extending from center to center of the transverse pile rows placed generally 10 ft. apart. This slab is designed to carry a live load of 500 lb. per sq. ft. for the 10-ft. span between pile rows, and is reinforced with ⅝-in. square steel rods. The latter run longitudinally of the pier, are 6 in. apart, and are staggered so that only alternate rods terminate on the same pile row, with ⅜ by ⅜-in. separating rods. The slab is of 1:2:4 Portland cement concrete, with ¾-in. broken stone, the upper ½ in. of the slab being of Portland cement mortar finished smooth. This rod reinforcement is intended to be standard, but the substitution of trade sizes of equal strength and efficiency is permitted, subject to approval.

Definite illustrations of this type of pier construction are found in the two new piers recently completed by the Department of Docks and Ferries at the Gowanus Section, South Brooklyn, one at the foot of

31st Street, 1 475 ft. long, and the second at the foot of 33d Street, 1 616 ft. long, each pier being 150 ft. wide. These piers are among the finest in the harbor, and are probably the largest of their type in the world. The unit cost is practically the same as that of the old wooden deck type. The decks have a crown of about 8 in., in order to shed the water. The inshore end of the concrete deck rests on the bulkhead wall, but is not attached thereto, a horizontal plane joint allowing the deck to slide on the wall as it expands or contracts on account of changes of temperature.

Twenty-six piers with concrete decks have been built by the Department during the past 7 years. The earlier type, as exemplified by the Chelsea Section piers, consists of a 6-in. concrete deck surface reinforced with expanded metal and laid directly on the deck planking. The next type produced omitted the deck plank, and is represented by eight piers with decks consisting of a concrete slab, 6½ in. thick, reinforced with expanded metal, the slab spanning yellow pine rangers running longitudinally of the piers and generally about 6 ft. apart.

The final type evolved, omitting the timber floor system entirely, and placing a concrete slab reinforced with longitudinal steel rods directly on the timber-capped transverse pile rows, is represented by eight piers, the most important examples being those at the foot of 31st and 33d Streets, South Brooklyn, and the Municipal Pier at Stapleton, Staten Island.

All these piers have been built where the condition of the river bottom underlying them was such that no settlement could occur, and they have behaved admirably. No repairs have been necessary, except to the fender system, and none are anticipated for many years to come, excepting the renewal here and there of an imperfect pile, where rot may appear above the water line. Such renewals can be made at a minimum of cost—a few dollars per pile—by bench-capping, without any interference whatever with the integrity of the reinforced deck itself.

Column Foundations.

For single-story sheds, where additional bearing strength is required in the new concrete deck pier for shed column or superstructure support, the question has been treated in general in the same manner

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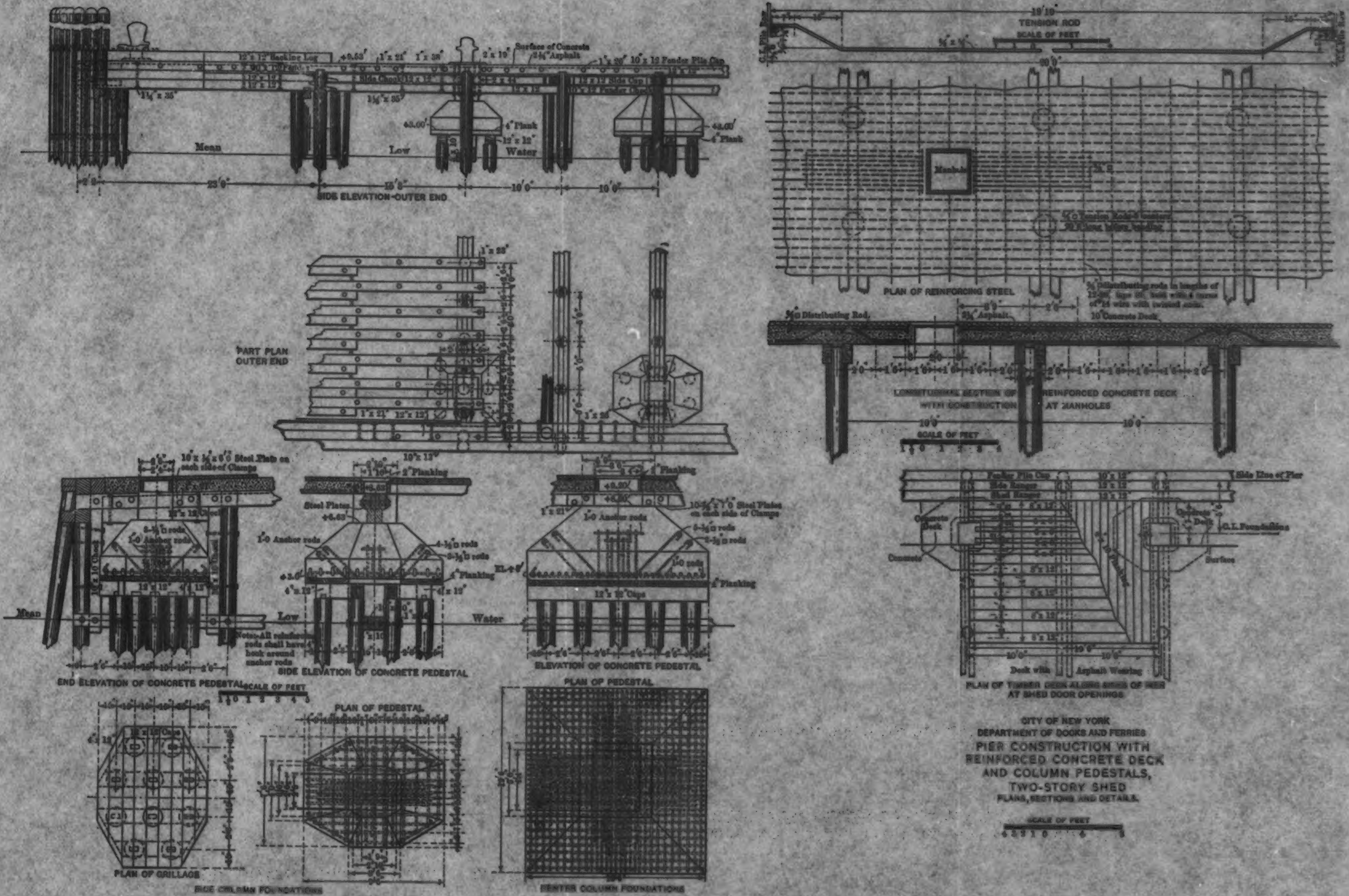


CITY OF NEW YORK
DEPARTMENT OF DOCKS AND FERRIES
PIER CONSTRUCTION WITH
REINFORCED CONCRETE DECK
AND COLUMN PEDESTALS,
TWO-STORY SHED
PLANS, SECTIONS AND DETAILS.

SCALE OF FEET

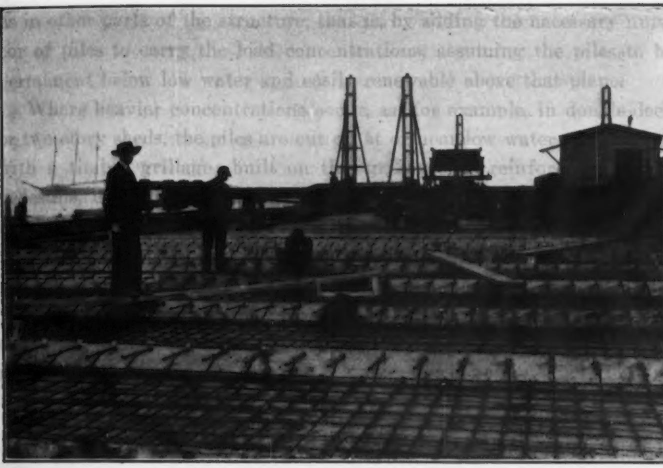
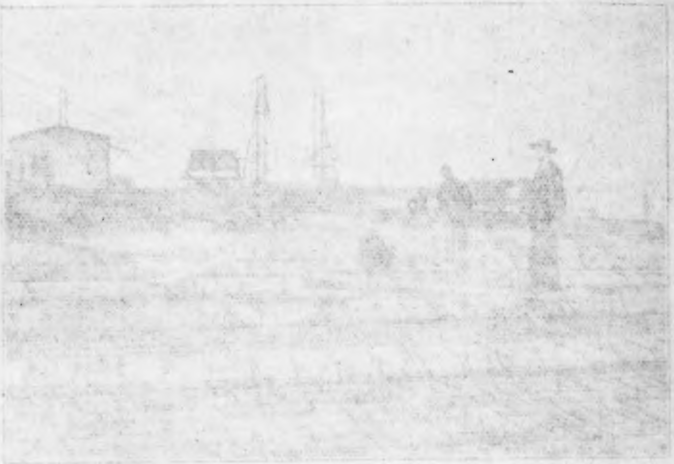
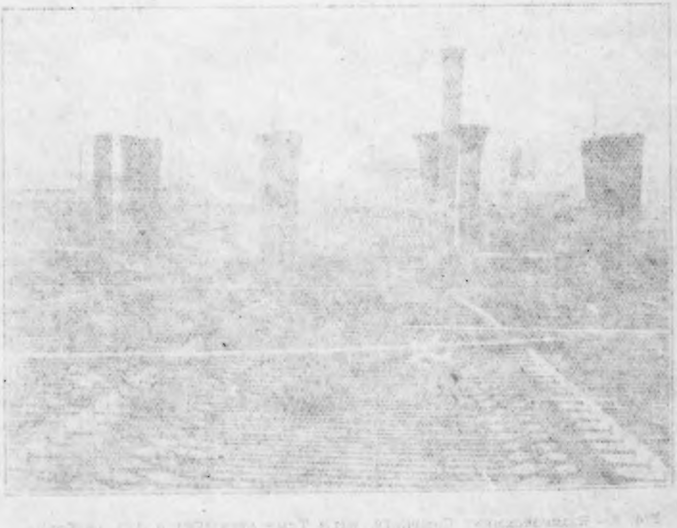


Fig. 3.—Tension and Distributing Reinforcement with Templates for Holding in Position.

Fig. 4.—Reinforcement Complete, with Templates Which Act as Forms for the Alternate Strips of Concrete Deck.





as in other parts of the structure, that is, by adding the necessary number of piles to carry the load concentrations, assuming the piles to be permanent below low water and easily renewable above that plane.

Where heavier concentrations occur, as, for example, in double-deck or two-story sheds, the piles are cut off at or near low water and covered with a timber grillage; built on this grillage are reinforced concrete pedestals, extending to the deck level, to carry the shed columns.

Railroad tracks, being a requirement on the South Brooklyn piers previously described, are carried on four lines of 15-in. steel I-beams, placed on the transverse clamp system of the pile rows and extending from the inshore end of the pier sheds to within 60 ft. of the outshore shed wall. The beams rest on steel saddles placed on the clamps, and are entirely encased in concrete.

TABLE 1.—COST OF CONSTRUCTION, MAINTENANCE AND REPAIRS.

Average Cost of Construction of Wooden Deck Piers, \$1.00 to \$1.15 per Sq. Ft.

REPAIR COSTS OF WOODEN DECK PIER.

Description.	Percentage of total original cost.	Renewal required.	Remarks.
Sheathing	12	Every 6 years.	
Backing log	1.8	Every 8 years	
Fender chocks, including vertical sheathing	4	Every 10 years.	
Fender piles	4.7	Every 12 years.	
Decking	11.3	Every 15 years.	
Bracing	7.1	50% in every 20 years.	
Rangers and caps	24.4	50% in every 20 years.	
Piles	34.7	33⅓% every 20 years.	Above M. L. W. only.

CONCRETE DECK PIER.

Cost of construction, 31st Street Pier,
South Brooklyn, no asphalt surface, \$0.87 per sq. ft.

Cost of construction, 33d Street Pier,
South Brooklyn, with asphalt surface, 0.97 " " "

Economy being a prime factor in its construction, it was decided to try out the concrete deck surface for wear and tear of heavy team traffic, and the earlier decks, therefore, were finished with a smooth mortar surface to receive this traffic. Two years of experimenting on these lines, determined the fact that though the concrete surface was

admirably adapted to light traffic, cargo handling by hand or motor trucks, etc., it could not stand the concentration of heavy team traffic confined within narrow lanes located generally in the center of the pier. The grinding and turning of heavily laden trucks inside these narrow lanes or zones gradually caused surface rupture of the top coat of mortar. It was decided, therefore, to place an asphalt wearing surface on the deck, and this has proven very effective.

The piers at the foot of 31st and 33d Streets, South Brooklyn, have been in service for about 3 years. No signs of cracking or other imperfections have appeared, and the piers as a whole are a complete success.

Repairs.

For the modern type of concrete deck pier, the cost of maintaining the fender system is about the same as that for the wooden pier; deck sheathing repairs are practically eliminated, except such minor asphalt patching as may be required, and can be considered negligible in a good asphalt deck under cover; the deck plank is eliminated; the life of the ranger and cap system is prolonged by the protection from moisture given by the impervious concrete deck, and the cost of maintenance and repairs, therefore, is reduced to a minimum.

Conclusions.

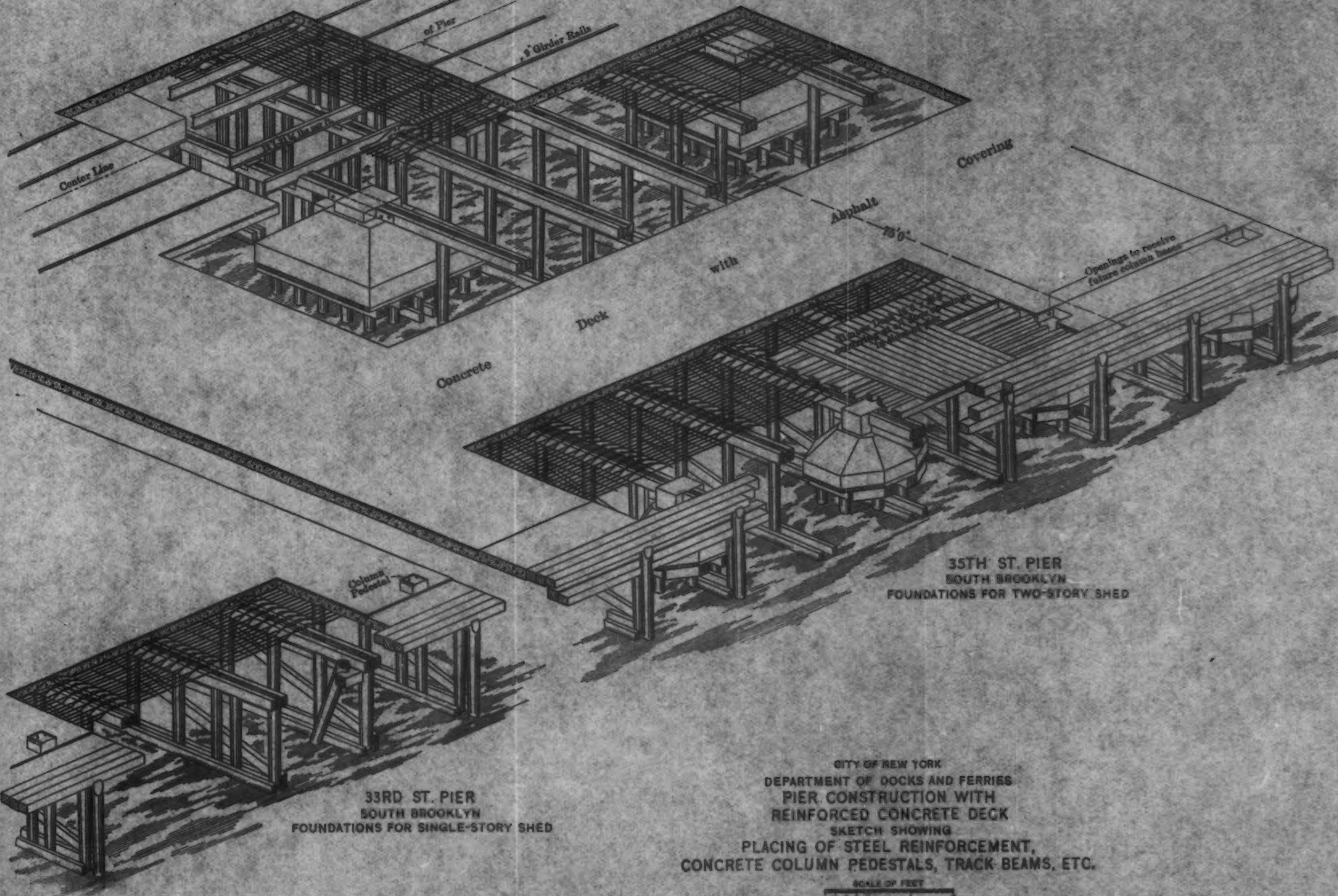
From the foregoing it will be observed that the problem which confronted the Department was the elimination of the timber deck and deck-supporting structure of the wooden pier, by the substitution therefor of some permanent form of construction meeting the following requirements:

(*a*) Economy in cost of construction and maintenance, the unit cost to be such as to produce or make possible a remunerative return on the capital invested.

(*b*) The construction to be of such character as to be readily extended, reconstructed, re-modeled, or, if necessary, entirely removed, as more intensive development of the area occupied by the pier or system of piers might be made necessary by the growth of commerce and shipping.

From what has been stated the following conclusions may be deduced:

PLATE VIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVII, No. 1292.
STANIFORD ON
PIER CONSTRUCTION
IN NEW YORK HARBOR.



CITY OF NEW YORK
DEPARTMENT OF DOCKS AND FERRIES
PIER CONSTRUCTION WITH
REINFORCED CONCRETE DECK
SKETCH SHOWING
PLACING OF STEEL REINFORCEMENT,
CONCRETE COLUMN PEDESTALS, TRACK BEAMS, ETC.

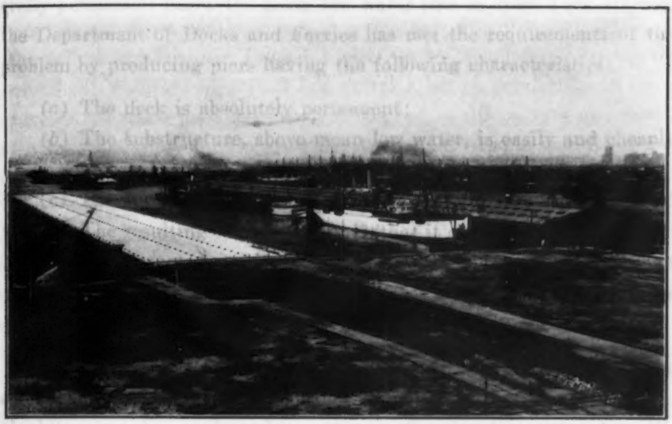


FIG. 5.—TWO PIERS WITH REINFORCED CONCRETE DECK, EACH 150 FT. WIDE. ONE 1 475 AND THE OTHER 1 616 FT. LONG.

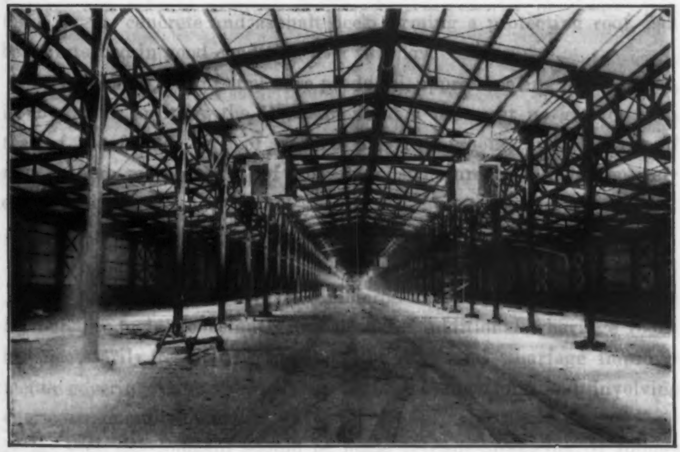


FIG. 6.—INTERIOR VIEW OF SHED ON PIER, 150 FT. WIDE AND 1 616 FT. LONG, SHOWING CONCRETE DECK AND TRACK STRINGERS, READY FOR RAILS.

First. Admitting that timber piles and foundation work are generally permanent below the mean low water line in New York Harbor, the Department of Docks and Ferries has met the requirements of the problem by producing piers having the following characteristics:

(*a*) The deck is absolutely permanent;

(*b*) The substructure, above mean low water, is easily and cheaply repaired and maintained;

(*c*) The supporting part, below the water line, is permanent; and

(*d*) The resulting structure is such that it can be readily extended, reconstructed, or, if necessary, entirely removed at a cost not prohibitive, as would be the case, for example, with most types of reinforced concrete deck-supporting structures.

Second. That the Department has produced permanent parts in the structures where these are essential. No attempt was made to obtain absolute permanency above low water, in the structure supporting the deck, for the reason that,

Third. This portion of the structure, the caps, piles, braces, etc., protected as they are from saturation by urine and other objectionable fluids by the concrete and asphalt deck forming a protecting roof, can be maintained in good condition at a very low cost.

Fourth. The type of structure produced, approximating permanency, is now being built by the Department at a first cost no greater than that of the former type of wooden pier throughout, and the cost of repairs and maintenance of the deck structure is almost entirely eliminated.

General Remarks.

The phenomenal growth of commerce and shipping in New York Harbor, producing constantly the need for additional wharfage room, has necessarily resulted in projects for dock and wharfage improvements covering extensive portions of the water-front and involving enormous amounts of money.

As such development should be made revenue-producing, it follows that a cheap type of pier construction, low in first cost, long of life, and economical to maintain, is one of the fundamental factors in determining the extent of the improvements undertaken not only by the City

of New York, but by private interests as well. This does not necessarily follow when smaller cities are considered.

Compared with the vast amount of money necessary for the comprehensive development of the port, the sums required for similar purposes by other ports on the Atlantic and Pacific seaboards of the United States are very small, in fact, in many cases, comparatively insignificant. Harbor improvements at these ports are generally confined to a limited area or extent of water-front, and consist usually of a few piers or bulkheads. Their aggregate cost is such that, even if not revenue-producing, the investment is often well within the financial capacity of the municipality and wisely undertaken by reason of its effect in stimulating existing trade and attracting new shipping business, producing what might be called "indirect revenue." This has often been the incentive in inaugurating harbor improvements at these minor ports, and it has resulted, in some instances, in absolutely permanent structures at such great expense that remunerative return on the capital invested is highly improbable, if not impossible; in other words, the benefits derived are being consummated at too great a cost, even when considered from the point of view of "indirect revenue."

Notwithstanding the many desirable features of this new type of pier construction, it has not been generally adopted, though many inquiries and requests for plans and descriptions of the type and method of construction have been received and replies made thereto, pointing out its proven economy in first cost, repairs, and maintenance. New York Harbor, however, presents a number of examples of similar types of concrete deck piers adopted by private interests, notable among which are: The reinforced concrete deck pier built by the Central Railroad of New Jersey at Communipaw, N. J., 892 ft. long and 131 ft. wide, used for railroad freight, tracks being laid on the pier deck; the pier under construction by the United States Lighthouse Board in the Reservation at St. George, Staten Island; and in two piers now under construction at the Brooklyn Navy Yard. The design of the latter, particularly, presents an admirable combination of reinforced concrete construction, surmounting the wooden pile and grillage supporting foundations near low water, resulting in a practically permanent pier at an economical first cost.

When reinforced concrete piers have been built at other ports along the coast, the attempt at absolute permanency, as stated above, has gen-

erally resulted in prohibitive first cost, when considered as a business proposition requiring interest on the capital invested.

Where the life of the wooden structure below the water is not endangered by wood-borers—in other words, where it can be considered as permanent—the difference in cost between an elaborate reinforced concrete pile and deck-supporting structure throughout, and a structure of the type herein described, represents practically that amount of money uselessly wasted.

Further, it is an open question whether the rigidity of a reinforced concrete structure throughout would not be a serious cause of deterioration in such structures, particularly below the water line. The shearing action of the impact of a large ocean-going steamship with a pier of this rigid type might cause a dangerous condition in the entire structure difficult to repair or overcome; even shocks tending to break the concrete surfaces of piles and substructure would admit sea water with its consequent destructive chemical action and freezing in cold weather.

On the other hand, the timber and pile substructure, with its flexibility and elasticity, acts, to a considerable extent, as a shock absorber, dissipating the effect on both the vessel and the pier, consequent on the impact of the collision or more or less violent contact with the pier, sometimes occurring when warping vessels into their berths.

The personnel of the Department of Docks and Ferries, as at present constituted, is as follows: Mr. R. A. C. Smith, Commissioner; Bureau of Engineering: The writer, Chief Engineer; S. W. Hoag, Jr., M. Am. Soc. C. E., Deputy Chief Engineer; and R. T. Betts, M. Am. Soc. C. E., Assistant Engineer, in charge of design.

DISCUSSION

Mr. Walker. E. G. WALKER, JUN. AM. SOC. C. E. (by letter).—The Engineering Staff of the Department of Docks and Ferries of New York City has evolved a novel and effective type of pier, without introducing any drastic departures from generally accepted methods of construction. Composite structures in which reinforced concrete is used are common, but a reinforced concrete decking on a timber pier, as detailed in the paper, is less common, and, in many cases, would need special justification by reason of the great difference which generally exists between the life of a concrete work and that of a timber structure set in water. In New York Harbor, however, there is a special combination of conditions, which, as a rule, does not occur elsewhere, and makes the composite piers described an admirable solution of the problem of providing wharfage space economically.

The figures in Table 1 show an appreciable saving in first cost by the adoption of the new construction, but no direct figures are given as to the relative maintenance costs. Inasmuch as reduction of the latter cost is often of greater importance than a small saving on first cost, the writer has endeavored to arrive at some relative figures from the data in the paper. From Table 1 he has obtained by deduction the figures in Table 2.

The author claims that, with the concrete and asphalt decking, repairs to sheathing and decking are practically eliminated, and that repairs to rangers and caps are reduced considerably. This gives the approximate figures in Table 3.

This total is only about 46% of the total annual cost of maintenance of the timber-decked piers, considering only equal capital expenditure. Inasmuch as the average cost per square foot for timber-decked piers is \$1.07, and for concrete-decked, \$0.97, it follows that the average annual costs per square foot for maintenance are:

For wooden deck piers.........\$1.07 $\times$ 0.0515 = \$0.055
For concrete deck piers........\$0.97 $\times$ 0.0234 = \$0.023,

showing a reduction of 58% by the use of concrete.

If this deduction is correct, it is obvious that a very great economy may be effected when large areas of timber piers have to be dealt with.

One feature of the composite pier construction has been omitted by the author, apparently owing to the fact that sufficient time has not yet elapsed to enable the requisite experience to be obtained. The necessity for fairly frequent reconstruction and removal of piers, or portions of them, is emphasized in the paper, but it is evident that the demolition of a reinforced concrete decking, though probably less troublesome than the removal of reinforced concrete piles and bracing, is still a considerably heavier and more expensive undertaking than

Mr. Walker.

the removal of timber decking. In cases where there is much chance of alteration of arrangements this might reduce very considerably the economy obtained by the use of composite piers. The writer would be glad to learn whether there are any data on this subject.

TABLE 2.—REPAIR COSTS OF WOODEN DECK PIERS.

Description.	Percentage of total original cost.	Number of years required for complete renewal.	Percentage of average annual cost of renewal.
Sheathing	12	6	2.00
Backing log	1.8	8	0.23
Fender chocks, including vertical sheathing	4	10	0.40
Fender piles	4.7	12	0.40
Decking	11.3	15	0.75
Bracing	7.1	40	0.18
Rangers and caps	24.4	40	0.61
Piles	34.7	60	0.58
Total annual cost of renewals, averaged over 60 years			5.15

TABLE 3.—REPAIR COSTS OF CONCRETE DECK PIERS.

Description.	Percentage of average annual cost of renewal.
Sheathing and decking	0.05 (say)
Backing log	0.23
Fender chocks, etc	0.40
Fender piles	0.40
Bracing	0.18
Rangers and caps	0.50 (say)
Piles	0.58
Total annual cost of renewals, averaged over 60 years	2.34

Possibly additional economy might be obtained by reducing the deterioration of the piles. In Table 3 it will be seen that renewals to piles form the largest item, namely, 0.58% per annum. It may be presumed that, as the destructive agency of marine boring insects is absent, the main deterioration of the piles is the decay which takes place on the alternately wet and dry surfaces between the extremes of tidal range. It might be worth considering whether a saving could not be effected by casing old piles in concrete for a sufficient length to prevent further decay. A casing of moulded concrete slabs. suitably reinforced and filled with concrete, can be fitted around a pile at very little cost, and is sufficient to retard almost indefinitely its deterioration. This method has proved effective in a number of instances.

Mr. Beugler.

EDWIN J. BEUGLER, M. AM. SOC. C. E. (by letter).—As literature on pier construction at American ports is somewhat limited, and engineers are frequently in the dark as to practice at the larger shipping centers, this paper will be appreciated by those having to do with the construction or maintenance of wharves and piers. American engineers have been criticized frequently for the adoption of what appears to be a temporary expedient in the way of shipping facilities, even at the larger ports. The analysis given in this paper will explain many of the questions as to why the particular type of pier construction commonly used in New York Harbor was adopted.

New York City is unique in regard to local conditions which favor the use of timber in the construction of piers. Though the cost of timber has been advancing from year to year, and will continue to do so until the figure reaches a point where other materials will have to be used from an economic standpoint, the substitution of concrete for the deck of the pier has enabled the cost per square foot to be kept within the limits of that of the former structure built entirely of wood. At the same time considerable economy has undoubtedly been secured, as far as maintenance costs are concerned.

It seems to the writer that still further economies in maintenance costs might be secured by the use of creosoted timber piles and caps, in order to avoid the renewal of the upper portion of the untreated piles and to maintain a fairly stiff structure. There is a possibility that bench-capping throughout the structure may weaken its resistance to impact from vessels striking the pier. Another advantage to be secured by the use of treated timber would be insurance against the action of sea worms.

Fortunately, at the present time, New York is surrounded by water, the impurities in which do not permit the *Teredo navalis* or other sea worms to exist. Conditions may change, however, and, in the East River particularly, much harm might result from the removal of sewage and other wastes which keep the water impure. The great quantity of fresh water in the North River would probably make the danger from worms more remote, even if the impurities were largely removed. The writer believes that this explains why timber piles are not affected in the immediate vicinity of New York. An experience which he had some years ago on railroad construction work along the Connecticut shore showed that yellow pine timber could not be kept intact much longer than one year. Certain piling was removed fourteen months after it had been driven and, with the exception of a core in the center of the piles about 4 in. in diameter, was found to be riddled by worms. This would seem to show that, but for the impurities in the water, the same thing would take place in the untreated timber piers in New York Harbor.

Mr. Beugler.

The adoption of all-concrete piers is probably quite remote for New York. The uncertainty as to the life and durability of plain or reinforced concrete piles in sea-water makes it inadvisable to invest a large amount of capital in such structures until their durability is proved beyond a doubt. At the present time it is believed that, if impermeable concrete can be secured, there will be little or no question of durability, but practically it is almost impossible to secure this class of work. The question of rigidity and the inability of a concrete structure to withstand shock seems to be a serious one, but not unsurmountable. Some of the concrete piers constructed along the Pacific Coast and elsewhere have special provision for absorbing the impact. One scheme, which appears to have worked fairly well, consists of a number of steel springs quite similar to those used in freight car construction, placed along the edge of the deck, with a long distributing timber instead of the usual fender construction. At other places the concrete construction has been faced with a line of spring piles arranged so that the piles absorb the shock before the bearing comes directly against the concrete deck. Even with the timber type of pier construction, it is quite necessary to secure a rigid structure, particularly where the pier is covered by a permanent superstructure of some size.

One interesting point brought out by the author is the development of the water-front from a distinctly business standpoint, that is, on the basis that all expenditures for improvements shall return a reasonable rate of interest on the investment. This has not always been the rule in the case of expenditures for public works, for which funds are apparently more easily secured than for private enterprises.

Mr. Taft.

HARRISON S. TAFT, ESQ. (by letter).—Mr. Staniford's invitation to present a written discussion of this valuable paper is accepted by the writer, perhaps with a little hesitation, as he feels that he may be "rushing in where angels fear to tread." A full discussion of the concrete dock problem would be so extensive and would lead into so many phases that it would be impossible to cover the subject thoroughly in a single article. Mr. Staniford treats of only one special phase of the problem. For several years the writer has been making a deep research and study of the whole subject, not only as developed in foreign countries, but as worked out thus far in domestic construction. The results of this research are being compiled in a rather extensive treatise or paper to be entitled "Concrete Docks," but as it is impossible to make the entire results of this research a part of this discussion, the writer will state simply a few of the facts, as therein recorded, in order to bring out the different phases of the subject.

Those who have studied the subject are no doubt aware that the first and most important question that confronts the engineer is the practicability of using concrete in sea water. There can be no denying the fact that, in attempting to make such use of con-

Mr. Taft. crete, there have been many disastrous failures. These have been due not as much to the design of the structure as to the use of unsuitable material and improper methods of placing the concrete, on which too much stress cannot be laid.

For structures in sea water, the whole subject reduces to the importance of obtaining a cement which possesses the inherent property of resisting the disintegration due to the chemical action of magnesium and the sulphate contents of ocean water on the alumina compounds in the cement; and also of obtaining an impermeable concrete without the use of any of the so-called water-proofing compounds. Such a cement must be low in alumina and high in silica; it should be of a slow-hardening but quick-setting type, and also of a fine pulverization, with no free lime. From the great success obtained by the use of foreign cements in structures placed in sea water, it is very evident that the German manufacturers solved this important question long ago. From the large number of various ideas of which one reads in American technical papers, it would appear that the manufacture of a domestic cement suitable for use in sea-water concrete is in the embryonic stage, the ideas as to the facts of the question being almost as numerous as the different brands of cement.

Although to the average concrete engineer the chemical composition and physical structure of the stone, sand, and gravel for use in proposed sea-water concrete structures would seem to be of minor consequence, it is a fact that such questions must be given the greatest consideration, if such concrete is to be of a lasting quality. It is of the utmost importance that such a concrete should have a maximum density. Thus the mechanical combination of the sand, stone, or gravel should be known, and modified to suit conditions as the work progresses. Far better results will be obtained by using a hard trap rock than gravel.

The next phase of the subject that confronts the engineer is the design. In designing a new type of structure one naturally investigates first what others have done along similar lines. Generally speaking, in America the problem is in a very early stage, as it is only natural that a forested country should be the last to take up the development of concrete docks. When the extensive reinforced concrete docks in England, France, Spain, and other European countries are investigated, it appears that this class of construction passed the experimental stage long ago. Especially is this so in England, where the art has reached a degree of perfection far in advance of that in any other country; such a degree, in fact, as to have become almost a standard for other parts of the world. A review of the English concrete docks shows that some have been in successful operation for eighteen years. Their maintenance cost is reported to be exceedingly small. In France, Spain, Italy, South America, and other foreign countries, concrete docks are

to be found, and their number is increasing rapidly. It is to be regretted that time will not here allow a detailed discussion of such an important part of the subject. Mr. Taft.

After the engineer has satisfied himself as to what has been done, the question of his own design stands out before him. There is no doubt in the writer's mind that the average concrete engineer is perfectly capable of designing concrete docks of one type or another which will more than fulfill the requirements for which they are intended; but, in making this design, the question is, what will be the cost of the dock? Thus the design problem reduces itself to a question of cost and the cost problem to a question of reducing to a minimum the quantity of concrete, reinforcement, and all other material, including forms, without diminishing the strength of the structure, an arrangement of beams, slab or otherwise, which will result in a most economical and practical method of construction.

It is unnecessary to impress on the members of this Society the fact that the cost of the design for a structure is but little in comparison with that of the material that goes into it and the necessary labor incident to its construction. As an axiom, the most economical form of concrete construction is manifestly that in which all secondary stresses are reduced to a minimum and all material is performing its utmost duty.

The next phase is, what type of construction shall be adopted for the concrete dock, in order to comply with the foregoing truisms? Shall the dock engineer recommend a full, out-and-out, reinforced concrete and structural steel dock, as worked out at San Francisco and later adopted by the United States Government for its concrete docks in the Philippines and Puget Sound Navy Yard? Or shall he recommend a driven concrete pile with concrete beam and slab-deck type of construction, so common in foreign countries and also found among American concrete docks to quite a large extent? On the other hand, he has the option of proposing a semi-concrete dock of either of two types: (1), wooden piling, plain, creosoted, or concrete protected, with a concrete deck; (2), reinforced concrete driven piles with a wooden decking. Right here Nature introduces another difficulty—the destructive teredo and other marine borers.

In fresh water it is perfectly feasible to adopt untreated wooden piles, with a concrete deck. In New York Harbor, as Mr. Staniford cites, this type of construction can be made a success because the sewage is destructive of the teredo in such waters; but, in the waters of the South Atlantic States, the Gulf States, the Caribbean Sea, and the whole Pacific Coast, conditions are met which have to be overcome by a different type of construction from that worked out so ingeniously under the author's supervision.

Mr. Taft. Another side of the subject is the comparison of wooden with concrete piles, with reference to load-carrying as well as cost and longevity. As a concrete pile will carry approximately three times as great a load as a wooden pile, and, under favorable conditions, can be put in place for about three times the cost of a creosoted wooden pile, would it not appear, in view of the possible long life of a concrete pile in sea water, that a semi-concrete dock—concrete piling and wooden decking—can be built at a reasonable cost compared with an out-and-out wooden dock? In other words, in teredo-infested waters, where a wooden pile dock has a life of about 15 years, a concrete pile dock will last, supposedly, for a whole generation, if not longer.

Though the chemical and engineering sides of the problem have their weight, the most important phase of the subject, in the final analysis, is finance. When a careful analysis of the relative value of initial cost, annual depreciation, fire insurance, bond issues, interest on bonds, and sinking funds is considered, the results are such as to astonish even those who perhaps may be well posted on other phases of the subject. Such an analysis will show that, whereas a wooden dock appears to be a most economical type with which to start, it soon becomes an expensive structure to keep in condition and carry all the overhead charges, especially if such a dock is built on a bond issue, the life of which exceeds the life of the dock, thus resulting in an overlapping bond issue in case the dock is rebuilt. The evils attached to such systems of financiering have been brought home most forcibly in connection with former paving work in New York City streets and with other public improvements. Until harbor development experts fully master the ins and outs of the financial side of the concrete *versus* wooden dock problem, they have another step to take in perfecting themselves in their special line of work.

Perhaps the most interesting phase of the subject, as the writer looks at it, is the construction. As Mr. Staniford has dealt entirely with the designing side of the problem, the writer does not feel at liberty to go into this phase of the question. It is one in which no doubt many engineers are deeply interested.

With this general review of the whole concrete dock problem before him, it was with deep interest that the writer read Mr. Staniford's paper and studied the standard type of construction his department has evolved from the old-fashioned wooden pile, wooden deck docks, a process of true elimination, with each step tested carefully before taking the next one. As the author states that twenty-six such piers have been built during the last 7 years, and adds that they have behaved admirably, it leaves only a small loop-hole for injecting any adverse criticisms of a first-hand nature, as it was during the later stages of the construction period of the Chelsea piers that the writer had an opportunity of witnessing dock work in New York City, although previ-

ous to that time he had been a resident of New York and vicinity for about 10 years. Mr. Taft.

In studying the author's design, the writer could not help comparing it with the so-called Cattle Dock at Liverpool, constructed of wooden piles and wooden bracing, but with a concrete beam and slab decking. As Mr. Staniford states that his type of semi-concrete dock has not been generally adopted by other ports, it would seem that dock engineers hesitate to build a permanent structure on a more or less temporary foundation, especially temporary in teredo-infested waters, in spite of the many excellent qualities claimed for this type, as respects the Port of New York.

When the life of a wooden structure below the high-water line is not endangered by wood-borers, it is quite true that a wooden-pile, concrete-deck dock can be built for a most economical figure; but, how many of our important harbors are free from the wood-boring pest? It would be of interest to know the probable cost of Mr. Staniford's type of dock if the wooden part of the structure were replaced with concrete piles and if a concrete cap were used.

A feature in Mr. Staniford's illustration that attracted the writer's attention was the thickness of the concrete deck-slab, as, in general, it appears to be so nearly that of a well-known flat slab system. On the principle that a dock is a structure carrying heavy floor loads, it is essential to adopt a substantial type of construction. Still, the writer would like to ask the author whether it would not be possible to make his dock still more economical and just as strong by using a shallow type of beam with a wider bent spacing, on the lines of the so-called "Floretyle construction," thus bringing the design into accord with the truisms and axioms previously stated? Or does he depend on his heavy concrete floor slab to obtain sufficient mass in his dock to help resist the lateral thrust from large steamships lying alongside? Not that the writer has any connection with the "Floretyle construction," but he is deeply interested in finding the greatest number of ways to reduce to a minimum the material needed in a concrete dock without decreasing its strength.

Another question: Would it not be possible to design a concrete pile dock with concrete caps and a shallow beam decking which would prove to be fully as economical as that shown on Plate VI, when such questions as depreciation, fire insurance, taxes, interest on investment, etc., are taken into consideration?

At first it would appear that the type of docks shown on Plate VI was lacking in cross-bracing, and that the lateral forces from the pounding of vessels would crack the concrete deck. As Mr. Staniford states that no cracks or other imperfections have as yet appeared, perhaps the heavy 10½-in. slab acts in such a way as to distribute any local lateral forces over a wide area. Still, the writer would be inter-

Mr. Taft. ested in seeing the results if one of the South Brooklyn docks were accidentally rammed by a steamer moving under one bell.

Mr. Staniford speaks of the objectionable feature of the rigidity in a complete concrete structure. If concrete possesses no elasticity, how does the flexibility of his wooden piles prevent the concrete deck from cracking under the force of a lateral blow from a vessel? As the author shows only a short length of piling on Plate VI, he gives no idea as to the depth of the water in which these semi-concrete docks are built, whether in shallow depth, with short unsupported lengths of piling and the slip dredged out, or whether the depth is uniform, such as 20, 30, 40, 60, 70, or even 90 ft., as in Seattle Harbor, and that, too, within 700 ft. of the bulkhead line. Such excessive depth might not give a dock of this type the same rigidity as obtained in the shallower harbors of the Atlantic Coast. Thus it might not be suitable for deep harbors.

Table 1 contains data of great value; it is the only statement of its kind the writer has found in his prolonged research. It would appear from this table that the average annual cost of maintenance and repairs for New York City docks is about 11% of their total cost. In Puget Sound waters, the annual depreciation of a creosoted pile dock is fixed by the valuation experts at 10% of its initial cost. In fresh water a depreciation of 5% is quoted. As good merchantable dock timber can be had, f. o. b. site, in Seattle for about $15 or $16 per thousand, with cement in carload lots at $2 per bbl. at the site, a comparison of cost of an Eastern dock with a lumber dock built in Puget Sound waters is perhaps not exactly equitable. Again, New York docks do not need concrete or treated piles as do those in Puget Sound. As the prices per square foot quoted by Mr. Staniford for the two South Brooklyn docks are practically the same as it is costing to build the new public docks (wooden creosoted piling and complete wooden decking) of Seattle for deep-draft ships, in view of the longevity of the New York type of concrete docks, the comparison is striking. It is the more remarkable because the under-water part of the South Brooklyn dock will last many decades longer than will the creosoted piles driven in the Puget Sound and other teredo-infested harbors.

Mr. Staniford states:

"The attempt at absolute permanency * * * has generally resulted in prohibitive first cost, when considered as a business proposition requiring interest on the capital invested."

He also says:

"It has resulted, in some instances, in absolutely permanent structures at such great expense that remunerative return on the capital invested is highly improbable, if not impossible; in other words, the benefits derived are being consummated at too great a cost, even when considered from the point of view of 'indirect revenue'."

It is perhaps true that when attempts have been made at absolute permanency at other ports than New York, the general result has been an excessive and perhaps prohibitive first cost, prohibitive in the past, except in our largest seaports. Still, full concrete docks have been built, and are still being built in some of our larger Pacific ports, as well as very extensively in foreign countries; but, what of the future? Has the concrete engineer reached the maximum economy in concrete dock design? If so, Mr. Staniford's statement can perhaps stand unchallenged. On this one point, if on no other, the writer desires to cross swords with the author of this valuable paper. It is a question which has been discussed frequently in the technical press of late, but all that discussion seems to be in connection with present and ordinary types of construction, and not along new lines of design—still trying to obtain economy by improving the reciprocating engine rather than by adopting the rotary type. Mr. Taft.

The writer does not wish to inject into this discussion anything that would detract from the interest and value of this paper; but, in compliance with a suggestion by Mr. Staniford, he submits some illustrations, indicating a method whereby it would appear that the maximum economy has not yet been reached in the author's design. An attempt will also be made to convince Mr. Staniford of the possibility of building a full concrete dock, the first cost of which will not be prohibitive. This will be done, not by filing a large number of plans covering various designs for concrete docks, but by submitting just one design wherein the quantity of material has been reduced to a minimum without any reduction in strength, as mentioned previously.

The most ordinary type of concrete construction is the well-known beam-slab system. In such a system the load on the floor is carried to the beams and thence to the supports. The upper three-eighths (approximately) of a concrete beam is in compression, the lower five-eighths (approximately) in tension. In designing reinforced structures, no account is ever taken of the tensile strength of concrete, the primary function of this material being to resist compressive and shear stresses. Steel is inserted to take up the tensile stresses. The concrete below the neutral axis is useful only as a means of holding the steel in position and making it act through shear. Thus it is apparent that more than one-half of the concrete in a beam is not utilized economically to its full capacity, and is therefore a considerable extra burden to the structure, resulting in an increased cost for material, forms, and supports. This waste becomes greater with increased spans and under heavy loading, especially over a water surface where expensive means of form supports are generally required.

During the past years a number of flat-slab types of construction have been brought out, all generic of the first idea, the so-called mushroom system; some of them are nothing more nor less than a shallow-

Mr. Taft. beam system masquerading under the name of flat slab. From reliable data furnished the writer, the thickness of the deck slab of these special types is so great as practically to counterbalance any material saving over a beam system, and it is an open question whether there is an actual cash saving in adopting such flat-slab systems.

Even if more economical, are these various flat-slab systems suited to dock work and the heavy lateral thrust that the docks have to stand? If not, is it possible to produce a new genera of concrete construction, either of a beam, a flat slab, or other type, that will stand in a class by itself and be suitable for dock work as well as for other heavy types of reinforced concrete construction? The writer's answer to the question is shown by Figs. 7, 8, and 9, of which only a brief description will be given at the present time. This system consists of large expanded hollow heads placed at the top of ordinary solid or hollow columns. The heads are of a conical or pyramidal shape, with a center post and diaphragms. Resting on the upper rim of the heads is the flat deck slab, sufficiently reinforced by non-trussed bars running diagonally, longitudinally, and crosswise, as may be necessary. The center post, the deck slab and the flaring sides of the heads form a perfect truss, integrated completely around the axis of the column. The heads should be poured some days in advance of the slab, the slab being poured after the heads have become well hardened. The necessity of binding the heads and the deck slab securely together will depend on the purpose of the structure, that is, whether it is to carry direct vertical loads only, or to absorb lateral shock in addition to a vertical load, as in dock work.

The heads may be likened to spread foundation work, made hollow and turned upside down, or the well-known household tin funnel resting on a firm foundation and supporting a slab or platform on its upper rim. The deck loads are supported by the expanded heads and transmitted through them to the columns and distributed on the piling or hard substratum by the enlarged base at the foot of each column.

This system does away with all the tension concrete below the neutral axis of a beam system. The flaring sides of the heads are in direct compression and shear. Thus all the concrete in the heads acts directly in taking up the stresses due to the superimposed load.

This system not only reduces to the absolute minimum the quantity of concrete needed, but also does away with trussed steel. This results in a structure far more economical than an out-and-out beam and slab system carrying heavy loads.

To absorb the lateral thrust on the dock, a suitable system of bracing of one type or another is provided, as shown. There is also a system of fender piles with their rebuffer springs, the lower spring supposedly absorbing the larger part of the lateral pounding and transmitting it directly to the bracing without any rocking of the heads.

Mr. Taft.

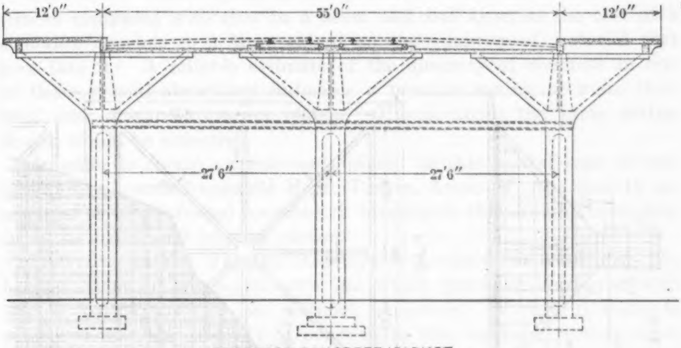


FIG. 7.

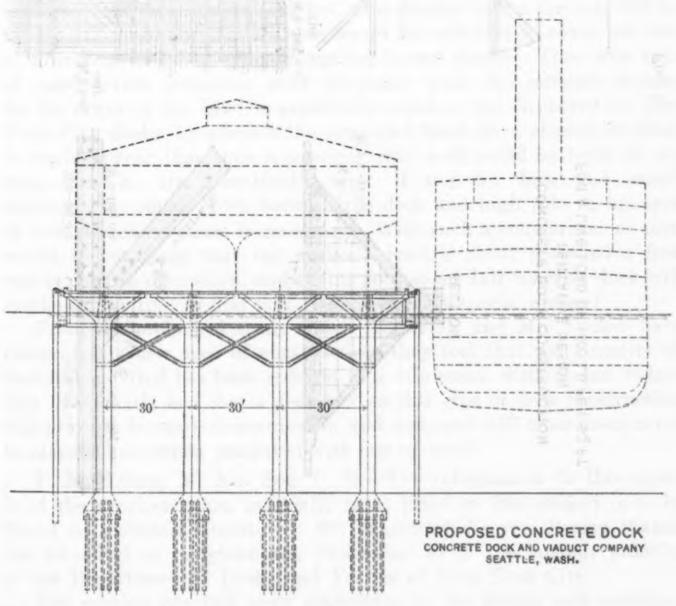


FIG. 8.

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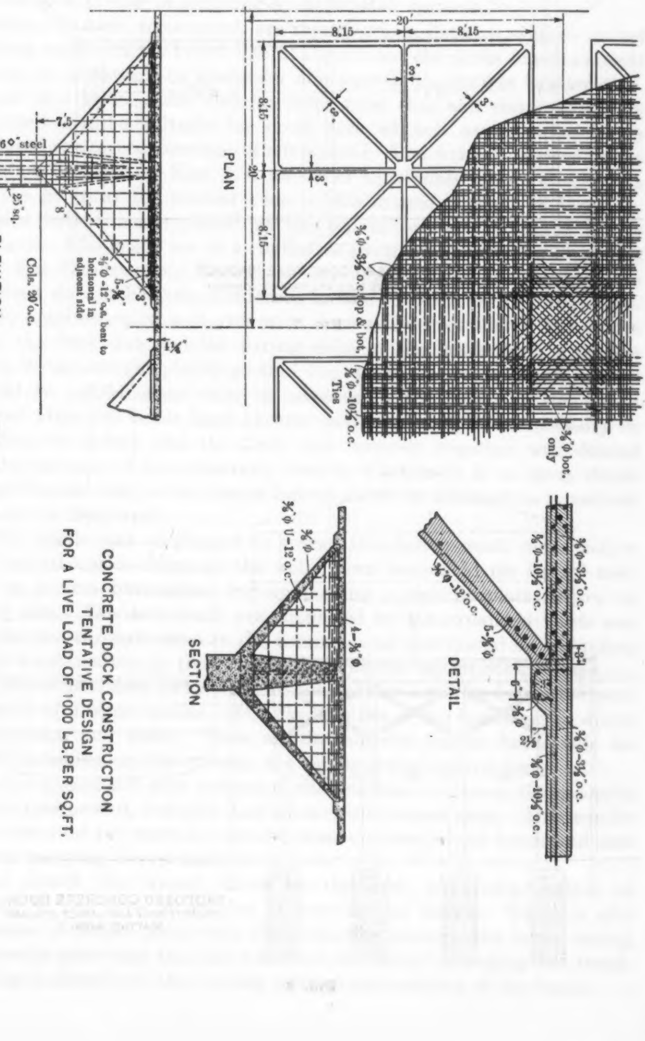

FIG. 9.

CONCRETE DOCK CONSTRUCTION
TENTATIVE DESIGN
FOR A LIVE LOAD OF 1000 LB. PER SQ.FT.

Just a few words regarding the quantity of concrete in such a system as compared with that in a beam and slab type, as the cost of a structure depends to a large extent on the quantity of material that goes into it: A reliable estimate of the quantity of concrete in one of these special structures indicates a possible saving of more than 20% over a standard beam system. Hence, about the same saving in cost would be expected. Mr. Taft.

In order to obtain an unbiased opinion on this special type of construction, the writer engaged H. F. Tucker, Assoc. M. Am. Soc. C. E., a specialist in reinforced concrete, to investigate this design thoroughly in all its many and unusual phases.

Referring to Fig. 9: The structure is designed for a uniform live load of 1 000 lb. per sq. ft. over the whole panel, as compared with 500 lb. per sq. ft. for the author's structure. A careful estimate will show that the quantity of concrete in this special structure for a 20 by 20-ft. panel, exclusive of the column part below the head, is practically 13 cu. yd. The quantity of concrete in Mr. Staniford's structure for the 20 by 20-ft., 10½-in. slab is the same. Inasmuch as the tentative design is for a direct vertical load of 1 000 lb. per sq. ft., perhaps it is not an equitable comparison; but, in a similar design for only 500 lb. per sq. ft., the saving in concrete would be sufficient to cover the cost of providing sufficient bracing against lateral forces. Thus this type of construction compares most favorably with the author's design. As the depth of the head is practically equal to the freeboard of New York City docks, by placing the expanded head on a cluster of piles, it would appear that such a semi-concrete dock could be built at the same cost as Mr. Staniford's type, if not for less, and would eliminate the repair work between the deck and high tide in his type of dock, which he states is necessary. With such a comparison as this, would it not seem that the author's remark about prohibitive first cost is open to discussion, and that a permanent full concrete dock will not be prohibitive in first cost if built on this special design?

From the prolonged study which the writer and Mr. Tucker have made of this new type of construction, they feel that the quantity of material required has been reduced to a minimum, without any reduction in strength, and that a dock having this type of deck construction will prove to be more economical in first cost, and still more economical in maintenance when compared with one of wood.

F. R. Harris, M. Am. Soc. C. E.—The information in this paper is of the greatest value, especially as so little on this subject is to be found in technical literature. Mr. Staniford deserves hearty thanks for his effort to enlighten the Profession as to the current practice of the Department of Docks and Ferries of New York City. Mr. Harris.

The speaker has had some experience in the design and construction of wharves and piers at various ports on the Atlantic Coast,

Mr. Harris. especially in New York Harbor, and feels that it is his duty to invite attention to certain parts of this paper, as his investigations have shown that the conclusions arrived at and implied by Mr. Staniford are not entirely beyond question and challenge.

About four years ago, the speaker made a thorough investigation of certain projected pier or wharf construction at the New York Navy Yard. Several such wharves had been built there, and, under a project for the development of the Yard and the improvement of the water-front, two more were projected, with the probability that these would be followed by the construction of five others. Another wharf, designed by the speaker, of the general type which had been used at the Navy Yard for some time, was just being completed.

Being familiar with the current practice of the Department of Docks and Ferries, he made a careful investigation of the design then used by that Department, particularly the concrete deck on timber piles and caps. It seemed especially desirable to use such a design on account of its reputed economy in first cost and its apparent advantage in being fire-retarding. The results of this investigation were not entirely in harmony with the conclusions set forth in this paper.

It was found that the cost of a concrete deck wharf would exceed that of a timber deck wharf, this conclusion being reached in spite of the contract price obtaining on the City piers at South Brooklyn. The speaker interviewed the contractor for some of these piers, and learned that the price at which they were being built was not a fair criterion, as the cost was considerably in excess of the sum the contractor was receiving. This served to confirm the estimates which had been made, particularly when the figures were taken up with various contractors. It became evident that, if bids were invited for piers or wharves with a concrete deck, and an alternate with a 4-in. deck and 3-in. sheathing on top of it, the cost of the concrete pier would be higher by about 25 cents per sq. ft. The cost of the substructure—the piles and caps—would be practically the same in both types, with a slight modification which will be mentioned later. The superstructure, which could be considered as the deck structure, would consist of rangers, 4-in. decking, and 3-in. sheathing, in the case of timber wharves; of 10½ in. of concrete and 2 in. of asphalt paving, in the concrete deck wharf. The lumber in the timber deck wharf would be about 8½ ft. b. m. per sq. ft., which, at $42 per thousand ($35 for lumber and $7 for labor), would amount to 35.7 cents per sq. ft. This should be compared with 0.03 cu. yd. of reinforced concrete and 1 sq. ft. of asphalt, in the concrete deck wharf, estimated at 60 cents per sq. ft., or about 25 cents per sq. ft. more than the timber deck wharf, which is an additional item in favor of the latter.

The deck structure of the concrete deck wharf would weigh practically 150 lb. per sq. ft., thereby decreasing its live-load carrying

capacity to that extent; the timber involved would only weigh 34 lb. per sq. ft., showing that the timber wharves would have a carrying capacity of 116 lb. per sq. ft. in excess of what the same foundation would carry with a different type of deck. Therefore, in order to make the two systems comparable, additional piles, and probably caps, would have to be provided in the concrete deck wharf, making the difference in cost even more favorable for the timber deck wharf. The speaker fully appreciates the fact that special conditions in taking care of the transportation and shipping of valuable materials in New York City involve fire risk and insurance, and that probably on this account the Department of Docks and Ferries was well warranted in substituting the concrete deck for the timber deck; and although this reason alone justifies its use, it is misleading to justify it on the basis of cost and economy. Mr. Harris.

As an objection to timber deck wharf construction, Mr. Staniford has mentioned the necessity of repairing and relaying the deck sheathing on account of its destruction by traffic. The speaker is also impelled to question the implication that this timber deck sheathing would have to be replaced more frequently than the asphalt wearing surface. From his experience with asphalt pavements, he is inclined to believe that a 3-in. deck sheathing would outlast the asphalt wearing surface, and—even if this conclusion were not entirely correct—that it would be cheaper to repair or replace the 3-in. deck sheathing than the asphalt pavement wearing surface.

Repairs is a very important item, when wharves are not of a permanent type, as is the case with those on timber piles and having either concrete or timber decks. As Mr. Staniford states, it is necessary, at intervals, to replace piles and caps, such repair being caused by decay from mean tide up, except, of course, in harbors where marine borers are found, where a timber pile wharf of any type cannot be considered in any way a permanent structure. The destruction of caps and stringers by rot requires serious consideration, especially as there is a tendency from year to year to deliver commercial timber of poorer quality, containing more sap. It is a well-known fact that inspection rules in relation to commercial lumber have become less and less rigid in recent years. A wooden deck wharf is not considered a permanent structure, and frequent repairs and replacement of the timber work and the piles are expected. These are readily made, as it is not difficult to remove the deck, replace caps, or drive additional piles. The speaker understands that a concrete deck wharf is considered by the Department of Docks and Ferries as more permanent, although why it should be he cannot comprehend, because it is apparent that the timber piles will need repair and replacement just as often as in the timber deck type; the pile caps also will need replacement just as often, and probably oftener, because the concrete slab will

Mr. Harris.

tend to keep the tops of these caps damp and in a condition to invite rapid decay. It will be extremely difficult to make repairs on one of these wharves. The speaker need not compare the task of removing timber planking with that of removing a 10½-in. reinforced concrete slab. The expense of cutting through such a slab, with its reinforcement, will be considerable. It is understood that wharves of this type have not been in use long enough to have required repair, but, when such repairs are necessary, there is no doubt that the difficulty will convince the Department of Docks and Ferries that a permanent deck structure on a temporary foundation is not the very best type to adopt.

There seems to be a generally accepted opinion that there are no marine borers in New York Harbor. This is an error. Marine borers are found in the Lower Harbor, and, perhaps, under certain conditions, which are likely to develop with wind and tide, they will be found on the South Brooklyn shore. A wharf of the Navy Department at Fort Lafayette, constructed about 10 or 12 years ago, was examined recently by the speaker, and preparations were made to repair it. It was found that in many instances the piles were entirely gone, having been destroyed by sea borers, and that the caps and stringers were in such a condition, on account of rot, that it would be an economy to replace the entire wharf with a new one.

In the investigation of pier construction in the vicinity of New York, with the idea of adopting a type for the Navy Yard, as previously stated, both kinds of wharves used by the Department of Docks and Ferries were considered carefully. Previous to that time, there had been used in the Navy Yard a type consisting of a pile platform at low water, surmounted at the sides and ends by masonry retaining walls, the contained space being filled with earth compacted and paved over. The last pier of this type—known as Pier D—designed and constructed under the speaker's supervision, cost about \$3.25 per sq. ft.; this included wood block paving on a concrete foundation, two lines of standard gauge railroad track and one line of 18-ft. gauge, 40-ton, crane track, and also fresh- and salt-water pipes, air pipes, and electric conduits. In making the study referred to, an endeavor was made to reduce this first cost and still obtain a substantially permanent structure. It is apparent, of course, that the pile foundation of such a wharf carries a very heavy dead load, namely, the masonry retaining walls and the 8 or 9 ft. of earth filling; or, in other words, that more than half the foundation piles are used in carrying a dead load, and to that extent are an absolute loss in so far as the live-load capacity of the wharf is concerned. The investigation and estimates established the fact that a timber wharf would cost about \$1.00 per sq. ft., and a timber wharf with a concrete deck, \$1.25 per sq. ft. The items of maintenance and repair

for each type were considerable, and in the case of the concrete deck, the difficulty of such maintenance and repair, entirely eliminated it from further consideration. The speaker, consequently, evolved a design in which he used a reinforced concrete deck, lighter than that of the Department of Docks and Ferries, but, to some extent, similar, and this was carried by reinforced concrete columns resting on a timber substructure. The latter consisted of piles cut off slightly above low-tide datum, and fastened to the cross-caps with oak tree-nails. The reinforced concrete columns were cast on shore. The bases were dovetailed, let into the caps, and wedged to them with spruce wedges. They were also fastended by stringers or clamps which were attached to the caps, and the space between the abutting longitudinal clamps was filled with 3 by 12-in. chocks or filling pieces. The usual brace piles at the side were used in the timber substructure. All the subaqueous work was fastened and wedged, as far as possible, oak tree-nails being used and metal fastenings avoided. The designs were of two types, one with girders and beams to support the concrete deck slabs, and the other using the mushroom system, each column unit being flared out so as to reinforce the criss-crossing of the main deck slab reinforcement. The preliminary estimates indicated that this mushroom type would be less expensive, but the bids received proved this to be in error, and, therefore, the girder-beam type was adopted. Two of these piers have been completed. They have a creosoted wood-block pavement over the concrete deck slab, two lines of standard gauge railroad track, one on each side of the wharf, and two lines of subway, one on each side of the wharf; with fresh- and salt-water pipes, pneumatic lines, and telephone and electric conduits for cables still to be placed. These piers were estimated to cost \$1.60 per sq. ft., but this was based on the use of piles 65 ft. long. On actually proceeding with the preparation of detailed plans, and an investigation of the site, it was found that piles 85 ft. long would be required, and this, together with the expense of inshore connections for pipe work, railroad tracks, etc., increased the price of the work, so that the actual cost of these two piers has been \$2.04 per sq. ft. After the experience gained in the construction of these piers, the speaker is convinced that piers of the size used in New York Harbor, and without the subway and pipe lines, track work, etc., could be constructed for less than \$1.60 per sq. ft. Mr. Harris.

It is apparent that piers of this type may safely be considered permanent, as far as concrete immersed in salt water is permanent. Every precaution which experience at the Navy Yard could suggest was taken to make this concrete work permanent, but so much difficulty has been had there with concrete in salt water as to suggest that this is not a permanent material in salt water. Dry Dock No. 2, at the New York Navy Yard, was originally built of timber, and

Mr. Harris. after very voluminous discussion before this Society, years ago, condemning timber structures of this sort, was rebuilt partly of concrete. It is now undergoing extensive repair. Considerable sums are being expended in replacing the facing of the concrete altars and floor, which has deteriorated and disintegrated to such an extent that it is possible to use a pick and shovel in removing the concrete.

The concrete unit columns in the wharves mentioned were cast on shore, a 1:2:4 mixture being used, containing a water-proofing compound. They were allowed to season for some time before being placed in the water.

The speaker's principal motive in discussing this paper is to call attention to the fact that the design used by the Department of Docks and Ferries is a composition of a permanent and a temporary structure. Special attention is called to the fact that it would be difficult or impossible to repair the temporary part—the foundation itself—without also destroying the permanent part, thus making the entire structure only temporary. He also desires to bring out the fact that, with slight additional expense, the permanent part of the structure could have been carried down to mean tide level, thus making the entire wharf permanent, as far as wharf structures of this character and of concrete can be permanent. In other words, in the upper part of New York Harbor, where timber piles will not be attacked by sea borers, and can be considered permanent from low tide down, the composition of timber and concrete, if made as described for the Navy Yard, furnishes a permanent structure at a low first cost. The speaker gladly acknowledges that the investigation made by him first brought to his attention the inconsistency in Mr. Staniford's design, and in an effort to retain its good features and at the same time avoid the expense of reinforced concrete piles, he succeeded in evolving the design he has described.

In reply to Mr. Snow: The speaker does not think that conditions in southern yards can be compared with those in the New York and Boston Yards. He believes that, in southern waters, concrete is free from deterioration partly because of the absence of extreme temperature changes which open up hair-like cracks in the surface, thus permitting the entry, or the attack, of salt water. It has also been stated that a gelatinous substance forms on the face of the concrete, between high and low water, and protects the surface. The speaker believes that even a very thin facing of granite will protect a structure from deterioration, although the joints, after a while, will require re-pointing.

Mr. Snow. J. P. Snow, M. Am. Soc. C. E.—There are many examples of concrete construction in sea water in Boston Harbor, and in every instance known to the speaker there has been serious disintegration be-

FIG. 10.—DISINTEGRATION OF CONCRETE, ABUTMENT "Q", VAUGHAN'S BRIDGE, PORTLAND, ME. ONE WINTER'S EXPOSURE. CONCRETE IN LOWER PART PROTECTED BY SAND. SLOPE OF SAND FILL SHOWN AT LOWER LEFT CORNER.

FIG. 11.—ABUTMENT "Q", VAUGHAN'S BRIDGE, PORTLAND, ME.

tween high and low tides. This trouble is attributed to the action of frost. The damage is readily repaired with a cement gun, if not allowed to become too extensive. It would be of great interest to ascertain if this sort of disintegration occurs in southern waters where there is no freezing, and the speaker would be glad to have Mr. Harris state, if he knows, whether Government works of this class suffer between high and low tides in southern harbors. Mr. Snow.

Mr. Shertzer. TYRRELL B. SHERTZER, ASSOC. M. AM. SOC. C. E.—In 1906, the concrete was placed for one of the abutments of a large highway bridge across the Fore River, a branch of the Harbor of Portland, Me. This abutment was constructed with a concrete base, granite facing, and concrete backing. The plans called for a sand-fill to be placed across the front and ends of the structure to a little above the elevation of the bottom of the granite, this fill to be paved so as to resist wave action. Conditions were such, however, that all the fill could not be placed before cold weather set in.

The winter of 1906-07 was a severe one, and it was noticed that the unprotected concrete base was disintegrating where exposed. In the spring of 1907, after the ice had gone out, the exposed surface was found to be disintegrated to a depth of more than 2 in. That portion of the concrete which had been protected by the sand, however, was found to be in perfect condition, and, at the elevation of the sand protection, there was a sharp and distinct limit to the area disintegrated. It will be seen clearly from Fig. 10 that the portion of the concrete below the sand line still shows the saw marks of the form boards, whereas that portion above the sand line is badly disintegrated. The small hole seen at the upper right-hand corner of the photograph was drilled for the purpose of ascertaining how deeply the concrete was affected. The slope of the edge of the sand-fill, which was excavated to permit of taking the photograph, may be seen at the lower left-hand corner. This photograph shows a portion of the same face of the base as appears in Fig. 11, the photograph of the entire abutment.

These conditions led to a series of experiments to ascertain, if possible, the causes of the phenomenon. In the fall of 1907, several sets of standard briquettes were made of both neat cement and of a 1:3 mix, making all the possible combinations of mixing with fresh and sea water, and using the local bank sand and standard Ottawa sand. Three 12-in. cubes were also made of samples taken from the regular machine-mixed concrete. These briquettes were made and placed at the beginning of cold weather in the winter of 1907-08. They were distributed as follows: Sets of three briquettes of each type and enough for the regular 7-day, 28-day, 3-month, 6-month, and 1-year tests were stored in the laboratory in fresh and sea water;

Mr. Shertzer. buried in the sand at a depth of about 1 ft. above high water, at mean tide, and below low water; and placed in crates above high water, at mean tide, and below low water. One cube was placed on the flats above high water, one at mean tide, and the third was lowered to the bottom of the river.

Chemical tests were made during these experiments by the professor of chemistry at the Portland High School to ascertain the effects of the materials and water.

Unfortunately, the long-time tests could not be made on the briquettes, as the City Hall at Portland was destroyed by fire early in 1908, and the laboratory and all the records and data were lost, so that what appears here is from memory only.

Such of the tests as were made showed that the briquettes stored in both fresh and sea water in the laboratory, those buried in the sand, and those exposed in the crates above and below water, gave practically identical results, and those stored in the crates at mean tide were entirely disintegrated after from 2 to 3 weeks of exposure to freezing weather.

The large cubes were made and placed in the spring of 1907, and were exposed throughout the summer of 1907 and the following winter. No effect was observed as the result of the summer exposure, and after about a year's exposure those cubes placed above and below water were still in perfect condition, the one exposed at mean tide, however, had been reduced to about 4 in. as the result of exposure during the winter.

In the spring of 1907, as soon as weather conditions permitted, the loose material was removed from the disintegrated face of the base of the abutment, the surface was plastered up, and the rest of the sand-fill and the pavement placed. In the fall of 1912 some of the rip-rap and sand protection was removed, and it was found that the trowel marks in the plaster used in smoothing up were still distinct. The sand lying within the tidal range has been saturated with sea water for more than 5 years, and it would seem that if the action were a chemical one it would have affected the concrete.

It would seem that the only conclusion which can be drawn from the above-mentioned experiments is that the disintegration was caused by purely mechanical means, and that if the concrete is protected from the direct action of alternate freezing and thawing, such as occurs within the tidal range, there will be no disintegration.

William H. Burr, M. Am. Soc. C. E., was the Consulting Engineer, and the speaker was Resident Engineer, on this work.

Mr. Stengel. C. H. Stengel, Assoc. M. Am. Soc. C. E.—In 1907, when the Virginian Railway coal terminals were constructed, it was necessary to build permanent foundations for the large steel superstructure, which

extended 1 000 ft. into the waters of Hampton Roads. Open pile foundations, capped with grillages, were out of the question, as the waters were infested with teredo. The design decided on was monolithic concrete piers, with heavy rectangular bases, built on piles in 30 ft. of water, the piles being cut off 1 ft. below the mud line. On the rectangular bases, battered pier sections were built to a level approximately 4 ft. above high water. All this concrete was deposited under water by tremies 12 in. in diameter, and allowed to set for several days; then the forms were removed and the sheeting was pulled. Mr. Stengel.

Under the conditions existing at the time, and on account of the manner in which this work was done, if there were any chemical action to deteriorate concrete, due to the effect of sea water, it would have been noticed, as a careful inspection of all the piers was made by divers; also, if there was any destruction between high and low water, due to changes in temperature and freezing, it would now be perceptible; as it is, these piers are in as good condition as when built.

The speaker, however, does not doubt that in piers built under similar conditions in northern harbors, where the temperature changes are considerable, and alternate freezing and thawing takes place between high and low water, some mechanical destructive effect might be experienced, as concrete built under the conditions stated never becomes dry, and the contained moisture, in freezing near the surface, would expand and gradually disintegrate the concrete.

This effect, however, should not obtain with concrete blocks, built and matured on land and then set in place between high and low water; and there are a number of examples around New York Harbor to substantiate this statement.

The mixture of concrete used in the construction of the foundation for the coal piers was 1 : 2½ : 5, with gravel.

L. D. Cornish, M. Am. Soc. C. E.—The question of the deterioration of concrete in sea water, naturally, was one of the first subjects of investigation by the designing force of the Isthmian Canal Commission in connection with the locks for the Panama Canal. Mr. Cornish.

Data on this subject were collected for about two years and, after careful consideration, the conclusion was finally reached that recorded experience failed to show that deterioration of concrete might be expected in tropical sea water, provided ordinary care was exercised in the selection of the ingredients, and in the mixing and placing.

Experience on the Isthmus, thus far, has shown no reason for a different conclusion, and the speaker is of the opinion that the causes for authenticated cases of deterioration must be sought in mechanical action or poor materials, rather than in chemical action of ordinary sea water on first-class American Portland cement concrete.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—Extensive harbor improvements call for strict economy in first cost and annual maintenance, and length of life. Recent developments on San Francisco Bay have given rise to some new features which may be interesting. The final conclusion is that a solid-mole development is much superior in every respect to the ordinary pier, when a wide view is taken of the whole situation from both financial and commercial standpoints.

As the waters of San Francisco Bay are largely infested with the marine pests, teredo and limnoria, all important structures are made of creosoted piles or concrete piles and superstructure.

First Case.—An Ordinary Pier.—For ample commercial purposes, the pier should be at least 200 ft. wide and 1 000 ft. long, with a slip alongside, 300 ft. wide, for handling the shipping, and, without any warehouses, tracks, derricks, movable cranes, etc., would cost probably $315 000.

Second Case.—A Solid Mole.—The mole would be surrounded on three sides with a 50-ft. commercial wharf. The same amount of money will build a solid mole, 500 ft. wide and 1 000 ft. long, and a slip alongside, 400 ft. wide, for handling vessels. This simple structure, like the pier mentioned, is supposed to be without warehouses, tracks, derricks, movable cranes, etc.

A little consideration will show the advantages of the mole development. The dock frontage of the pier development is 2 500 lin. ft.; that of the mole development is 2 900 lin. ft.—a difference of 16% in favor of the latter. In the case of the mole the area of dock flooring to be kept in efficient repair is only about 60% of that of the pier development, which is quite an important item of annual expense. Besides these advantages, the mole has nearly twice as much warehouse room, and a 200-ft. roadway in the center for railway and general commercial traffic, the paramount convenience of which cannot be overestimated.

Mr. Davis. Chandler Davis, M. Am. Soc. C. E. (by letter).—New York Harbor, without doubt, is the most important one in the world. It possesses many advantages over other seaports, not the least of which is the small range of tide. As tidal and wet basins are unknown, these expensive constructions are eliminated from the problem. The current in the river is not so strong that the largest ships cannot be warped into their slips without much difficulty, although at times it may require several hours—it took 5 hours to dock the steamship *France*. Such cases, however, do not occur so frequently that the pier and slip system should be condemned, for it is, without doubt, the best and most economical layout which could have been adopted. It permits the handling of more ships in a given length of river-front than any other plan.

If the scheme adopted at Antwerp had been introduced at New York, a steamship of the dimensions of the *Imperator,* would require about 1 000 lin. ft. of bulkhead for docking purposes; with the plan used in New York Harbor this length of river-front is sufficient to permit the working of four such ships simultaneously. The late General McClellan, the first Chief Engineer of the Dock Department, was wise in developing and perfecting the system he found in use. **Mr. Davis.**

The Hudson River has been encroached on to such an extent that the War Department has finally decided that the pier-head line must not be moved farther out into the river, and all encroachment must be stopped. This attitude of the Government is well taken. When one considers the size of the modern ship, he will realize that, if such extension were permitted, there would soon be insufficient width of clear water to allow the safe handling of these vessels, for at least two ship lengths are required to turn them. At the present time there is barely the necessary width left between the pier-heads, and there is great difficulty in handling the ocean liners. If relief is required, it must be sought by moving the bulkheads inland or locating the docks at points other than on the Manhattan shore of the Hudson River.

The naval architect is increasing not only the length of ocean liners, but also their beam. This is a very important factor, and must be considered in laying out new work. Fig. 12 shows the average increase in beam of ships, for every five years from 1865 to date; it will be seen that it has been more than doubled. The *Labrador,* launched in 1865, had a beam of 39.2 ft.; the new Hamburg-American liner, *Vaterland,* will have a beam of about 100 ft., or two and one-half times that of the *Labrador.*

The open water between two adjoining piers, called the "slip", should be of sufficient width to allow the working of two steamers in the same slip at the same time. The following formula will determine the required width, provided railroad car-floats are not used, as is the case with fruit ships:

$$W = 2F + 4C + 2B + 2L + 50.$$

W = the required width of slip;
F = the width to be allowed for the fender system of the piers, including the rolling log, if one is used;
C = the extreme width of the coal barges;
B = the beam of the ship;
L = the extreme width of the cargo lighters; all expressed in feet.

In the center of the slip, a width of 50 ft. of open water is required to facilitate the handling of the lighters and scows without interfering with the working of the ship.

Mr. Davis.

The general tendency is to sacrifice the slip in order to build large, imposing piers. A width of 90 or 100 ft. for a pier is ample to care for the cargo of the largest ships afloat, and it is quite unnecessary to exceed this, unless it is intended to use such piers as store- or warehouses.

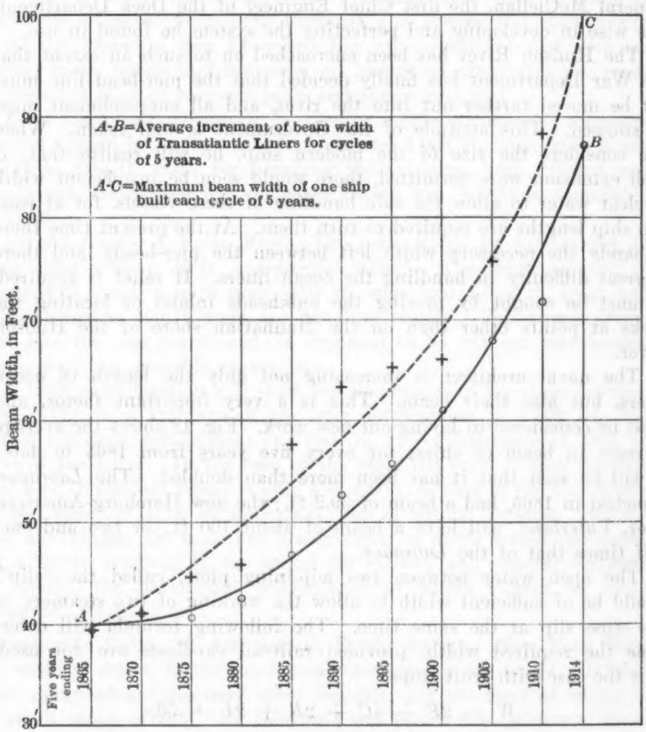

FIG. 12.

Ships of the *Imperator* or *Vaterland* type, having a beam of about 100 ft., would require slips about 400 ft. wide, if two such ships are to be docked in the slip at the same time. These points should be carefully considered in laying out a plan for docks, as congested slips increase the cost of handling cargoes and coaling. The dimensions of the largest vessels trading with the port must be considered in the design, and a study must be made of their gradual increase in size.

The materials which enter into pier construction are many. For a long time only wood was used, but recently the wooden decks and rangers have been replaced with concrete. The North German Lloyd Steamship Company was the pioneer in such work in this part of the world. After the disastrous fire which destroyed its Hoboken docks, the plant was rebuilt with concrete decks and fire-proof superstructures. This scheme has been still further developed by the City of New York, and to-day the construction above the caps or clamps is of reinforced concrete. This construction, however, leaves the piers vulnerable between low tide and the deck. The Hudson River is free from the *Teredo navalis* and other wood-destroying borers, and will probably remain so until some other method is found of disposing of the tons of filth which are daily dumped into the river. The piles below low water, therefore, are, for all practical purposes, indestructible. The few points where protection against worms has been found necessary need not be considered here. In all sections of the port, however, the piles above mean low water will deteriorate and eventually will have to be replaced. The present design of decking used by the City will make such work expensive and extremely difficult. The design, therefore, should include the entire pier from low water up, if it should be desirable to continue the use of wooden piles. A general plan for this could probably be worked out, and would only need to be modified to suit local conditions. Mr. Davis.

The Dock Department is working along the right lines, and is attacking the problems, as they present themselves, in a conservative, but thorough manner.

R. T. BETTS, M. AM. SOC. C. E.—In the type of pier for single-story sheds, developed and built successfully by the Department of Docks under Mr. Staniford's direction, concrete is used for the deck only. Therefore, there can be no question about its durability, as far as exposure to sea water is concerned. In the type of pier for two-story sheds, pedestals of mass concrete, deposited *in situ* on a pile and timber foundation, will support the shed columns. With regard to the durability and permanency of Portland cement concrete exposed to the action of sea water, to which Mr. Harris has referred, the speaker would be inclined to condemn the use of concrete for such exposure, if the experience of the Department of Docks had been as unfortunate as that of the Government, as exemplified by the sea-wall at the New York Navy Yard, and elsewhere. Mr. Betts.

Since the establishment of the Department in 1871, Portland cement concrete has been used in block form and in mass, in the construction of the bulkhead or sea-wall. For the most part the wall built along the shores of Manhattan Island consists of large concrete blocks made in the proportion of 1 part cement, 2 parts sand,

Mr. Betts. and 5 parts $2\frac{1}{2}$-in. broken stone, by volume, and reaching on the face of the wall, from a prepared foundation of concrete in bags on rock bottom at a depth of from 15 to 40 ft. below mean low water, or from a pile foundation sawed to a grade of about 15 ft. below mean low water, up to about $2\frac{1}{2}$ ft. below mean low water, the remainder of the face, up to the street grade, consisting, usually, of granite backed with concrete. In the fabrication of these concrete base blocks no attempt whatever was made to produce a dense face, except the usual "spading" along the form. They were allowed to harden thoroughly in air, and their use has resulted in permanent construction. About 5 or 6 months ago, two of these base blocks were removed from the wall at the foot of East 38th Street, East River, in order to enable the New York Edison Company to place an additional intake tunnel. An examination of these blocks, after a submergence of 10 years, showed perfect surfaces. There can be no doubt as to the permanency of Portland cement concrete when moulded in blocks and allowed to harden in air before being exposed to the action of sea water.

Numerous examples (many of them built by the Department of Docks) of concrete deposited *in situ* and exposed to tidal action, such as concrete walls on rock, concrete walls on pile and timber platforms, concrete walls as facing for timber cribs, and concrete foundations for ferry buildings, exist in and about New York Harbor. A personal examination of many of these structures has shown that in nearly every case where deterioration is found, it has occurred in a zone extending from about mean low water to about mean tide, and, as might be expected, walls exposed to considerable wave action show greater deterioration within this zone. It is also observed that where the precaution has been taken to face up the wall with a richer (1 part cement to 2 parts sand) and denser mixture, from 3 to 6 in. thick (as in the case of the Whale Creek wall, and the Wallabout wall adjacent to the New York Navy Yard), the exposed faces are intact, although it must be stated that the Wallabout wall was not exposed to tidal action until after the concrete had had time to become well hardened.

From the nature and location of the deterioration, there can be but little doubt that it is due in the beginning to a dislodgement of the cement by the action of the water before it has had time to set, and its further progress is due to mechanical action, such as frost, ice, and impact from floating débris, rather than to chemical action. In order to prevent spreading, repairs are made, cheaply, easily, and effectively, by cutting out the affected places to a depth of about 2 in. on a falling tide and replacing them with a mixture of 1 part cement to 2 parts sand.

The speaker has no hesitation in saying that good and permanent mass concrete, exposed to the action of sea water, can be obtained, provided the usual, fundamental precautions for concrete work are observed, as follows: Mr. Betts.

(*A*)—*Design.*—All surfaces of concrete deposited *in situ* and exposed to water should be composed of a dense, rich mixture of 1 part cement to 2 parts sand, from 3 to 6 in. thick, depending on whether or not it is exposed to wave action in addition to frost, ice, and floating débris. All edges should be rounded or beveled.

(*B*)—*Proper Materials.—Cement, Sand, Stone, and Water.*—The Department specifications for cement require that it "shall not set within half an hour after being mixed with water and shall set within five hours. * * * The fineness shall be such that the percentage of cement passing through a No. 100 sieve shall not be less than 95%." The tensile strength as developed by briquettes of neat cement should be 500 lb. per sq. in. for 1 day in air and 6 days in water, and 575 lb. per sq. in. for 1 day in air and 27 days in water. For a mixture of 1 part cement to 2 parts standard quartz sand, the values at the before-mentioned times should be 225 and 300 lb. per sq. in., respectively. The cement is subjected to the usual "pat" tests for checking, cracking, distortion, and color.

The sand should be clean and sharp.

The stone should be clean, hard, and durable. Gravel should not be used as a substitute for broken stone, except possibly, in "backing" concrete, where it will not be exposed to tidal action.

The water should be fresh and clean.

(*C*)—*Workmanship.*—All ingredients should be mixed thoroughly so as to form a "wet" mixture, and care should be exercised in depositing it so as to get a uniform distribution of the ingredients, without "pockets." "Facing" should be deposited simultaneously with the "backing" and in such a way as to make the mass monolithic. Concrete, especially the exposed surfaces, should not be deposited in water, but the work should be planned and executed so as to keep the depositing in advance of a rising tide. Forms should be carefully made, preferably of dressed, tongued-and-grooved lumber, treated with crude oil or similar material to prevent adhesion of the concrete, and made tight enough (caulking the joints if necessary) to exclude water from the moulds, especially on a rising tide. They should be designed and braced so as to retain their shape accurately, even should it become necessary to fill a mould rapidly.

All concrete surfaces should be kept wet with fresh water for at least a week, in order to insure the proper initial hardening of the cement.

In hot weather, if exposed to the direct rays of the sun, the work should be covered with tarpaulin or similar covering and kept moist to prevent "drying out."

Mr. Betts.

In freezing weather the concrete materials should be heated, not only for the immediate purpose of mixing, but for keeping the frost out of the materials as well. The water should be kept hot, but not boiling; and all concrete deposited during freezing temperatures should be immediately covered and protected from frost until setting is assured.

Walls of mass concrete, requiring the depositing of concrete under water within forms, should not be attempted, unless the enclosing structure or forms are allowed to remain and become a part of the permanent construction, as it is practically impossible to obtain proper surfaces with concrete thus deposited.

Mr. Staniford.

CHARLES W. STANIFORD, M. AM. SOC. C. E. (by letter).—Some of the discussion has wandered into questions not germane to the subject matter of the paper, and has entered into matters pertaining to harbor design and general methods of construction.

The particular object of the paper was simply to show what had been accomplished by the Department of Docks in producing something more permanent than the ordinary New York wooden pier, and at about the same cost.

It is the writer's opinion, however, that all the discussions are valuable, as they bring out many admirable points in connection with harbor design and general designs for actual construction.

It is repeated that the old wooden pier, as built in New York Harbor up to 10 years ago, because it was easily removed and reconstructed, was one of the greatest factors in developing the harbor facilities. Instead of the shipping being compelled to adapt itself to the location and operation of massive permanent structures located along the water-front, as in English and Continental harbors, where there are stone quays and piers, the structures for berthing vessels and taking care of cargoes in New York Harbor were located to meet the conditions of shipping as these conditions developed and increased facilities were required. As the size of the vessels increased, the docks grew larger, in width as well as length, in proportion to the width of the slips, that is, the distance between piers increased to accommodate the greater beam of ocean steamers. It can readily be seen that if the New York water-front was lined with a system of massive and permanent stone, steel, or concrete structures, the development of the harbor could only proceed at enormous expense, and consequently with much less rapidity than is made possible by building structures of lighter form. The type formerly built by the Department met for the time all requirements, but, at little added expense, many objectionable features of the former construction have been eliminated, thus doing away with constant repairs; and, for the present, at least, the Department has a type which is durable and at

the same time inexpensive, and admirably meets the present-day demands of shipping. The result of this more modern method of construction has been shown in the paper, and the figures substantiating the statements are based on fact, not conjecture. Mr. Staniford.

Mr. Harris questions the lasting qualities of the asphalt wearing surface, and infers that a 3-in. deck sheathing would outlast the asphalt. The experience of the Department with deck sheathing is absolutely as cited in the paper, and not a matter of inference. Repairs to New York piers are absolutely needed at short intervals, resulting in frequent renewal of the whole deck sheathing, the wear on the latter being the result of a traffic movement probably unequaled in the world. In fact, this whole paper deals with conditions of this kind, where structures are under intense use constantly, and not with docks built in Navy Yards or in any place for public function, where team traffic is almost unknown, or where the docks are practically intended as a berthing place for steamers while undergoing repairs.

As a concrete illustration of the inexpensiveness of the asphalt wearing surface, the piers of the Chelsea Section may be cited. On these nine piers, each approximately 800 ft. long and 125 ft. wide, asphalt wearing surfaces were laid, and for a period of 5 years, under the most extreme traffic conditions, no repairs were necessary. Under the contract for this work, 96 989 sq. yd. were laid, and a bond for 5 years was given to maintain the asphalt wearing surface; at the expiration of this 5-year bond period, repairs were only needed on 780 sq. yd., costing $800, or less than 1% of the total cost. From the expiration of the bond period to the present day no repairs of these surfaces have been necessary; and, furthermore, no repairs are needed at the present time. Fourteen other piers may be cited where the same result has been secured by the use of asphalt.

As stated in the paper, for light traffic, this asphalt wearing surface may be eliminated, but, for heavy traffic, it has been found absolutely necessary to put some deck wearing surface over the concrete; and in some other localities the writer has advocated wood block or brick pavement for this purpose. Granite block pavement has been considered, but was not adopted because it would add so much to the weight.

Mr. Harris questions the facility of making repairs to decks of this kind. These repairs, like all others relating to dock and harbor structures, must be made by specialists in work of this class. The Department force, consisting of twenty pile-drivers with their crews, is constantly engaged in doing just such work.

It is mentioned that the eventual destruction of caps and stringers by continuous repairs deserves consideration. This is just what was considered in the design of piers of this style. By eliminating all the stringers and all the deck, leaving practically little more than the

Mr. Staniford.

timber caps, the quantity of timber to be renewed is reduced to an absolute minimum, the life of the caps, in addition being prolonged considerably by the use of the new system with its water-proof deck. In time, when repairs to this composite structure are required, owing to the rot of this insignificant quantity of timber underneath, such repairs will not necessitate the disturbance of the concrete deck, as all the work may be done below it by simply supporting the structure between alternate pile rows, removing the caps, and bench-capping the piles, the usual method by which similar repairs have been made by the Department for 25 years or more.

TABLE 4.—PIERS ON WHICH ASPHALT SURFACES HAVE BEEN ADOPTED.

Location.	Area, in square yards.	Price per square yard.	Elapsed time, in months.	Area repaired, in square yards.	Percentage of area, in stated time.	Percentage of area repaired per year.
Piers 53 to 62, N. R.	96 989	$0.90	34	40	......	......
		......	3	70	......	......
		......	8	175	......	......
		......	13	280	......	......
		......	2	145	......	......
		......	5	70	......	......
		Total:	60	780	0.8	0.16
Pier, W. 40th Street, N. R.	2 817	$1.48	48	none.	......	......
" 5, E. R.	4 315	1.19	30	none.	......	......
" 6, E. R.	3 043	1.19	12	25	......	......
			9	50	......	......
		Total:	21	75	2.5	1.43
Pier 22, E. R.	2 785	$1.48	60	20	0.7	0.14
" E. 5th Street, E. R.	2 876	1.18	22	25	1.0	0.55
" E. 20th Street, E. R.	3 602	1.49	46	70	2.0	0.52
" E. 95th Street, E. R.	1 614	0.90	27	none.	......	......
" E. 100th Street, E. R.	1 095	0.90	28	none.	......	......
" E. 110th Street, H. R.	1 964	1.49	48	none.	......	......
" Canal Street, Stapleton.	2 296	1.23	60	20	1.0	0.20
Platform, Stapleton Ferry	1 594	2.40	60	35	2.2	0.22
Pier, 52d Street, Brooklyn	2 719	0.84	23	25	1.0	0.52
" Whale Creek, Brooklyn	4 788	0.98	16	none.	......	......
" Jamaica Avenue, Queens.	742	1.45	18	none.	......	......

The repair of an occasional pile in the interior of the pier, on account of rot, is accomplished without any trouble by cutting the pile and replacing the head by bench-capping or splicing, thus making the structure as strong as the original. This has been done in a number of instances in New York, even under elaborate buildings, such as ferry houses, without any trouble whatever, and without any interruption of traffic.

The design of this pier offers relief from constant repairs, and makes unnecessary the expense of actual reconstruction from mean low water up, for the experience with the asphalt deck surface has covered a sufficient time to prove that repairs have been greatly eliminated by this feature, and the structure is actually preserved by the asphalt deck surface. Mr. Staniford.

The writer gave what he considered to be full recognition to the design of the piers being built in the Brooklyn Navy Yard, as being a good type for producing an absolutely permanent structure in these waters. To produce anything in the line of permanent development, however, is simply a question of expense. Mr. Harris has given some figures relative to the cost of piers in South Brooklyn, and has inferred that these prices were not a fair criterion, it being contended that the piers were built at extremely low cost, and with no profit to the contractor. This contention is difficult to prove. In the paper, the cost data for these piers are correct, and are repeated and elaborated as follows: The 31st Street Pier was let to the lowest bidder at $174 587. This was one of twelve bids, the next four higher being: $183 820, $192 462, $195 897, and $196 725. The 33d Street Pier was let to the lowest bidder at $236 500. This was one of seven bids, the next three higher being: $244 587, $246 332, and $254 897.

In each case the bidders for these piers were among the most reliable dock-building contractors in New York; and without question, any one of them could have performed the work successfully for the amount of his bid.

The New York Department of Docks has developed many designs for absolutely permanent structures, not only on the lines of the piers in the Brooklyn Navy Yard, made permanent from low water up, but absolutely permanent structures with concrete piles, and various forms of filled-in piers, some of which are without doubt as good as those cited. In any event, no two designs are exactly the same. Where the money is available, and the cost of deck repairs will necessarily be slight, and where a permanent structure is warranted, Mr. Harris is unquestionably right in suggesting that the structure should be made permanent from low water up.

Mr. Harris states that the generally accepted opinion that there are no marine borers in New York Harbor is in error. He states that they are found in the Lower Harbor, and that perhaps under certain conditions they may develop with wind and tide on the South Brooklyn shore. The writer fails to see why he cites the fact that he had found sea borers at Fort Lafayette. This question of the presence of the teredo, limnoria, or other wood-boring animals was settled by the Department more than 40 years ago. In fact, it was not only settled by the Department, but by all the corporate interests which have helped to build up New York Harbor while spending several hun-

Mr. Staniford. dred millions of dollars in building wooden structures. Unquestionably, if New York had remained a settlement with none of the evils attendant on the growth of a city of even ordinary size, these wood borers would have remained in the harbor, and the present form of construction would not have obtained. This has not been the fact, however, and 30 years ago the last evidences of the activity of these animals disappeared in all pier construction built either by the City or by others in the Upper Harbor.

In presenting this pier as a type of construction for New York Harbor, the writer took it for granted that it would be understood as applying to piers to be built in the harbor proper, and not for outlying sections nearer the ocean, where, of course, the teredo exists, this form of sea life requiring clear and fresh salt water in order to live. These wood borers have long been driven from the harbor proper. Even 10 years ago, the Department, in approaching the zone of danger (at the end of the Lower Bay) adopted the plan of creosoting all piles, but even in such localities, namely, along the northerly shores of Staten Island, at St. George, and other places where the teredo had been somewhat active, untreated piles which had been in the water for 8 years have been pulled and have shown absolutely no sign of any wood borers.

Before determining to use timber in the South Brooklyn Section, the Department for a number of years made a most exhaustive investigation, with divers, of hundreds of existing piers, bulkheads, and other structures along the water-front, looking for signs of infection by the teredo. The result showed that, although this sea worm had been somewhat active there in years past, it is now practically extinct.

Furthermore, for years past, the Department has been continually repairing the older structures built by the City. In the course of these repairs, the renewal of piles, even in the older structures, is found necessary only at rare intervals in the interior of the piers where they have been protected by the timber deck; though near the sides, where exposure is greater, more extensive repairs are needed. Also, the Department is continually removing old structures, some of them the oldest in New York Harbor, to make way for improvements. In these removals every pile is examined, under orders, for any evidence of the teredo, in order to obtain some idea of the time when it disappeared from that particular locality. In some instances the piles show that they were attacked by the teredo in the olden days, but, probably on account of increasing pollution of the water, it had ceased to exist. Piles removed from structures built within the past 25 years are as good as the day they were driven, and are constantly used again in other work.

Recently, the Department has been removing some of the old Brooklyn ferry houses on the East River, these structures being among the

very oldest on the water-front. In pulling the piles, each was examined with particular care to see if the teredo had made any ravages, but no evidence was found in any case. Mr. Staniford.

The following is a concrete example of the wonderful preservation of these piles and the non-existence of the teredo. When the old Pennsylvania Ferry House, at the foot of Cortlandt Street, one of the oldest structures along the lower water-front near the Battery, was being removed, the writer visited the premises in order to determine whether it would be advisable to use some of the old piles, which happened to be in the right position to take their place as part of the grillage for the concrete retaining wall which was to run under the new ferry house. He found that the piles, cut several feet above low water, were in a remarkably good state of preservation; the foreman, on being told that the piles were as good as the day they were placed, replied, "They are better than the day they were driven." They were subsequently retained and remained in the permanent structure as specimens of absolutely perfect piles, thus saving the cost of placing a number of new ones.

Mr. Snow says:

"There are many examples of concrete construction in sea water in Boston Harbor, and in every instance known to the speaker there has been serious disintegration between high and low tides."

In New York this condition does not obtain in the concrete works produced by the Department of Docks. A few other concrete bulkheads have disintegrated in this way, notably at the Brooklyn Navy Yard, where such a bulkhead, approximately a mile in length, has entirely disintegrated and been removed. The last piece of this concrete wall is now being demolished near the end of the Cob Dock, where absolute disintegration has taken place throughout, extending many feet back from the face, including the front piles, ends of caps, etc.

It is presumed that the Government will replace this wall with a new concrete structure, and then all the old Navy Yard concrete wall will have been reconstructed.

Up to the present time there has been no failure in any part of more than $8\frac{1}{2}$ miles of concrete and granite wall construction, built by the Department of Docks and Ferries, during a period of more than 40 years.

Frank W. Hodgdon, M. Am. Soc. C. E., Chief Engineer of the Massachusetts Board of Harbor and Land Commissioners, writes as follows:*

"Practically all the structures built of concrete in Boston Harbor have been badly disintegrated between high and low-water marks."

* *Engineering Record*, June 10th, 1911, p. 655.

Mr. Staniford.

To this the writer replied as follows:*

"That it will be of interest to your readers is evidenced by the fact that this office is in continual receipt of letters of inquiry from all parts of the world, not only from harbor engineers, but from engineers engaged in the design of bridges in order to ascertain the condition of such work built of concrete in salt water, in view of the failure of such structures throughout the world.

"It certainly is a question concerning which engineers should receive all information possible whereby people at large may receive due consideration in assuming the financial responsibility in future for constructions of this kind which are supposed to be built with a view to permanency.

"I will not attempt to give the history of anything but city construction work in New York Harbor, and the condition of same at the present time. Most of the work done here during the past forty years has been done by the Department of Docks, or what is now known as the Department of Docks and Ferries of the City of New York. During this period the city has been engaged in the construction of a permanent sea-wall, located principally around lower Manhattan, below 80th Street. This has been added to recently by quite extensive construction at North Brother Island, in the Borough of the Bronx, at Whale Creek in the Borough of Brooklyn, and at the Gowanus Section, South Brooklyn.

"This sea-wall in a general way varies in type of construction, according to the character of foundation, but the essential features above the foundation are very similar in all types. Almost every type of bottom is encountered in New York Harbor, while the wall has been built to meet these varying conditions, about twenty-five different modifications having been used thus far in the foundation work to meet the conditions imposed. There are three general types, however; one built upon solid rock, one built upon hard bottom, such as good, stiff clay or compact sand, and one built on soft river mud, forming a wall supported by mud flotation.

"In many parts of the waterfront of Manhattan, for instance, bed-rock is over 200 ft. below the river bottom, overlaid with soft mud. In the rock sections the wall has acquired a height above the rock as much as 50 ft. In other rock sections the wall is founded upon rock that has had to be blasted in order to create a sufficient depth for wharfage in front, and to take off sufficient rock for a stepped-off bottom in order to safely found the wall against the slope of the natural rock.

"In the soft mud bottoms the longest piles are used. In most types requiring pile foundations, the piles are usually cut off at a depth of about 15 ft., below mean low water, and from the top of these piles the wall proper is built.

"In the case of the rock bottom type the wall proper may be considered to commence at the top of the bag foundation which is built immediately on the rock bottom to a proper grade, for receiving the wall proper. From the tops of these pile and concrete bag foundations, the wall is built up to the vicinity of mean low water of concrete blocks, varying according to design from about 20 to about 95 tons

* *Engineering Record*, August 5th, 1911, p. 176.

CONCRETE BLOCKS BUILT AND PLACED IN WALL BY DEPARTMENT OF DOCKS, AND REMOVED BY THE EDISON COMPANY IN THE CONSTRUCTION OF INTAKE TUNNELS.

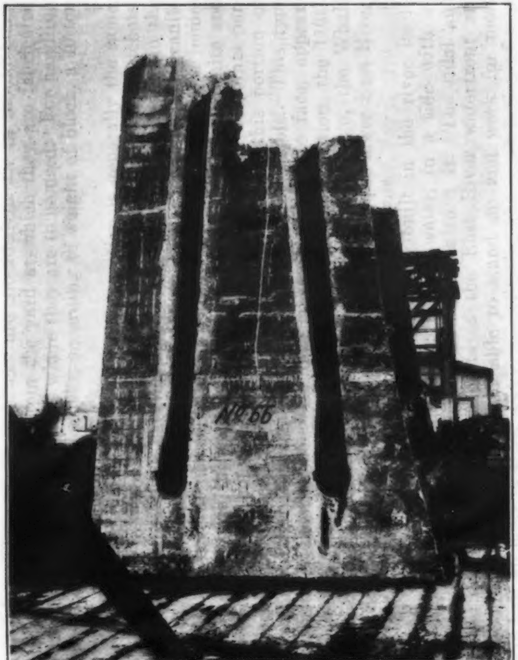

FIG. 13.—END VIEW.

FIG. 14.—BACK VIEW.

CONCRETE BLOCKS BUILT AND PLACED IN WALL BY DEPARTMENT OF DOCKS, AND REMOVED BY THE COMPANY IN THE CONSTRUCTION OF HUDSON TUNNELS.

FIG. 11.—BACK VIEW.

FIG. 12.—END VIEW.

Mr. Staniford.

in weight. These concrete blocks are moulded in air in mould boxes and are shipped on scows from the yard at which they are fabricated to any point on the waterfront where they are to be used. For handling these blocks we use two derricks, according to weight of block, a 40-ton derrick and a 100-ton derrick.

"Above these blocks the type of construction is generally the same, consisting of four courses of granite facing, of headers and stretchers, dove-tailed into a thick backing of concrete deposited en masse in the rear of the granite, and all carried up to the grade for the granite coping. In several places concrete deposited in situ forming a homogeneous mass with the concrete backing is substituted for granite and carried up to the coping course. This is usually done at points particularly where piers project from the bulkhead, as this portion of the wall is covered by the pier and, therefore, not visible. This type of construction with concrete face, instead of granite face, appears also in the general construction of the Claremont Section, the 116th Street Section, the Gowanus Section in South Brooklyn, the Whale Creek Section in Brooklyn, the Fulton Section on the lower East River, and part of the Old Slip Section on the East River.

"All of this wall construction has been built in the river itself, generally well off from the shore, in salt water, in a tide with an average range of 5 ft., varying in extremes to 8 ft. The tidal and current conditions in most parts of the East River waterfront are so severe that a diver is not able to stand up and work for more than half an hour in slack water, necessitating the construction of temporary breakwater barriers as a protection for the diver and for work against the current.

"The Department of Docks and Ferries, up to the present time, has built about 8½ miles of this river or sea-wall, requiring in its construction in the neighborhood of 6 000 concrete blocks. In the course of years the Department has had many opportunities for examining the condition of this wall in connection with the building of sewers, etc. Up to the present time no disintegration has been discovered that can be attributed to the existence of the structure in salt water. The concrete itself is in an admirable state of preservation, absolutely hard, and is undergoing no regular process of disintegration. In some cases between high and low water marks, where the concrete has been deposited in situ, there are indications of ice abrasions, but nothing whatever to cause any softness or to expect ultimate trouble.

"In some cases there are indications of the effect of wave action or scour on the freshly deposited concrete within the tidal range, due to carelessness in work on the mould boards, creating opportunity for tidal or wave scour, but this trouble does not appear to be progressive after the original scouring action on the freshly deposited concrete, excepting in the case of gravel concrete. It is my opinion that gravel concrete is not a proper material to use en masse (not moulded in air), where it is to be subjected to wave action or tidal effect in the process of depositing.

"Up to the present time the Department has not been obliged to spend much money on repairs to the sea-wall. This wall was built to be permanent, and at the present day all of the exposed concrete, which in the course of a few years may become a little worse from

Mr. Staniford. abrasion than at the present time, can be repaired and brought to its original state with the expenditure of a few hundred dollars.

"The extreme conditions that exist in Boston and that are to be seen in certain parts of this harbor, outside of work done by the city, and, in fact, in all of the various harbors of the world, could not exist and continue in the municipal work of the City of New York under its present financial control and government.

"Concrete harbor works built in other cities under municipal, private or corporate control, as well as constructions in European harbors, were, of course, all built originally as improvements, at great expense, over timber construction which might have been built under ordinary expenditure at a far less cost, and are only permanent investments, according to their behavior; therefore, because of the partial failure or threatened failure of so much concrete work in harbor construction, leading engineers throughout the world are assuming unusually expensive construction in their endeavors to produce permanency, by going to the other extreme and using granite or stone.

"Notwithstanding the failure or partial failure of concrete in the presence of salt water, in certain localities, the fact remains that this sea-wall which has been under construction by the City of New York for 41 years, is at the present time an excellent piece of work and is subject to the same climatic conditions as all cities on the Northern Atlantic Coast with the attendant ice, cold and rain characteristic of this latitude."

With reference to the concrete blocks mentioned by Mr. Betts as having been removed from the bulkhead wall after immersion in the salt water of the harbor for 10 years, Figs. 13 and 14 are submitted. These are from photographs of the blocks after they had been removed from the wall at 38th Street, East River, and placed on the Department Dock at the East 24th Street Yard. These blocks, weighing about 80 tons each, and recorded as Nos. 66 and 82, respectively, were fabricated on June 20th and August 16th, 1902, and were set in the East 38th Street bulkhead wall on July 14th and August 30th, 1902. They were removed from the wall by the Edison Company, in the construction of intake tunnels, on October 31st, 1912.

The abrasions along the top and parts of the bottom edges of these blocks are the result of chiseling, in taking the blocks out of the wall and breaking their contact with the concrete backing and granite which surmounted them. They are in a perfect state of preservation, showing no disintegration, even to the extent of the displacement of a particle. The edges are practically knife-edges, and the sharp-cut ridges along the joint lines of the mould boards show clearly. These blocks will be used in other work of the Dock Department when convenient.

Mr. Taft has presented a design for a permanent concrete dock which no doubt would answer for some localities where the bottom would permit of constructing a foundation such as he recommends. It is novel in many of its features, and has many good points; but the

question of decreasing the thickness of slab in the type of deck now being built by this Department would not be considered here. This is a question that cannot be decided analytically, but must be answered by experience with structures subject to the heavy side shocks which piers receive, together with the shocks from traffic, and from merchandise falling while being handled on the pier. The slab described in the paper has been tried out and its thickness is slightly greater than that used in the original piers; any future change, if ever made, would be to increase it. Mr. Staniford.

It may be interesting to many of the members of the Society to mention that another governing principle with the Dock Department engineers, in evolving a type of pier leading to the use of a reinforced concrete deck, was the elimination of any possible patented process. Municipal contracts in New York must be prepared so as to avoid the use of any patented processes, in order that competitive bids may be received to produce the structure under the design presented. This is not only a barrier, but is troublesome throughout the whole preparation of the contract and while the work is under construction. The Department, therefore, has evolved a type which, at least, has not caused any trouble from this source; it simply calls for a prescribed size of rod and spacing, or their equivalents in other shapes, on which a bid can be made by anybody manufacturing steel for reinforced concrete.

Mr. Taft's type, however, has many good features, and might well be used by private corporations desiring a permanent structure, the writer's only objection to it being the thin slab for heavy traffic. This is not such a serious factor as to detract from its usefulness, as the necessary increase in the thickness of this slab would not add greatly to the cost, although Mr. Taft makes quite a feature of this.

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TRANSACTIONS

Paper No. 1293

FLOOD FLOWS.*

By Weston E. Fuller, M. Am. Soc. C. E.

With Discussion by Messrs. Arthur E. Morgan, H. V. Hinckley, E. F. Chandler, Allen Hazen, Morris Knowles, Herbert E. Bellamy, E. Kuichling, Robert E. Horton, G. B. Pillsbury, and Weston E. Fuller.

The determination of the magnitude of the maximum flood that may be expected to occur on a river is of importance in many problems connected with engineering works. The problem is a complex one because floods are produced by combinations of a large number of conditions, many of which are themselves variable quantities.

Floods which have occurred on some rivers have been greatly in excess of others on the same river. These great floods come rarely on any one river, but an extraordinary flood occurs on some river every year. A study of their past frequency gives the best indication of what may be expected in the future.

The object of this paper is: 1st, to present a study of the frequency of floods; 2d, to show the relation between the catchment area and the magnitude of the flood; and 3d, to present formulas and tables to serve as an aid to judgment in estimating the probable maximum flood to be expected on a river.

Formulas for problems of this nature should be considered only as convenient and simple means of expressing the data. They serve as a framework on which to arrange the data in an orderly manner, so that they can be better understood and more readily used. The

* Presented at the meeting of October 15th, 1913, held at New Orleans, La.

formulas are of value only when accompanied by clear and concise tables of data. The tables here presented contain coefficients deduced from the data for many streams. These coefficients show the flood-producing capacity of the streams. Coefficients are given for streams in various parts of the country, and should prove of service in selecting coefficients for others for which local data are not available.

The conditions which produce or affect floods may be divided into two classes: First, those which relate to one stream and tend to make all floods on it greater or less than on others; and second, those which are general in their effect, as far as area is concerned, but are variable in time, tending to produce floods of various magnitudes from time to time.

In the first class may be included the following: The prevailing conditions of rainfall; the size, shape, and slope of the catchment area; the character of the soil and vegetation on the catchment area; the physical characteristics of the stream channel; the storage capacity in reservoirs; and many other physical characteristics of the catchment area and the stream itself. Some of these characteristics may be changed in time by the action of the elements or by the works of man; otherwise, their effect on floods may be considered as constant.

In the second class may be included the following: The rate of rainfall; the snow conditions; the temperature conditions; the quantity of water stored in reservoirs, lakes, and ground at the time the flood occurs; the velocity and direction of the storm; the formation of ice dams or other temporary obstructions in the river; and the many other elements which cause one flood to differ from another on the same stream.

No two floods are exactly alike. Two storms of like intensity, velocity, and direction passing over a catchment area may produce different floods. One coming at a time when the water in the ground and in the lakes and reservoirs is low may produce only a moderate flood. A second, coming at a time when the lakes, reservoirs, and ground are filled, may produce a large flood; or, if the second occurs in conjunction with high temperature, when there is a large quantity of snow on the water-shed, a very large flood may occur.

When the great variety of conditions which affect each flood is considered, it will be seen that the number of combinations which may occur is infinite. When many conditions tending to large floods occur coincidently with great rainfall, extraordinary floods are produced.

The chances of such a coincidence may be considered equal for different streams. Clearly, the study of the effect of elements of this class is one in probabilities.

A study based solely on the larger floods would be misleading, as a flood of a size which would be ordinary on one stream may be extraordinary on another stream. It is essential, then, to study each river separately, and to establish some standard which represents a normal flood on that river before the study of the probability of the occurrence of extraordinary floods is undertaken.

The standard selected for this use in this paper is the average yearly flood. By using this average flood, the effect of the variable elements is neutralized to a large extent, so that a direct comparison of one river with another is possible. Also, a comparison of the larger floods with the average flood on the same stream gives a means of determining the frequency with which the former occur.

To give a clearer idea of the scope of the paper and of the methods used, a few definitions, a summary of the more important conclusions, and a statement of the formulas proposed are here given.

Unit of Flood Measurements.—All quantities of flood flow are stated in cubic feet per second.

Twenty-four-Hour Average Flood.—Unless otherwise stated, all floods are on the basis of the average rate of flow for 24 consecutive hours. Most of the data available are on this basis, which is the standard used in publications of the United States Geological Survey.

Maximum Rate of Flood.—As in many engineering problems it is important to know the maximum rate of flow of the flood, a separate study is made to show the relation between the maximum rate of flow and the 24-hour average rate of flow.

Yearly Average Flood.—The yearly average flood of a stream is obtained by selecting the maximum 24-hour average flood for each year and taking the average of these floods.

Frequency of Occurrence of Floods.—By frequency is meant the probable time interval, in years, between the occurrence of floods of approximately the same magnitude.

Nomenclature.—

Q = the greatest average rate of flow for 24 consecutive hours during a period of years, in cubic feet per second;

Q (Max.) = the maximum rate of discharge of a flood, in cubic feet per second;

Q (Ave.) = the average yearly flood, in cubic feet per second;

T = the number of years in the period considered;

A = the catchment area of the river, in square miles;

C = a coefficient which is constant for the river at the point of observation.

Formula Proposed, as Obtained by Plottings of the Existing Data on American Rivers.—

$Q = C\,A^{0.8}\,(1 + 0.8 \log. T)$, in which Q is the largest 24-hour average rate of flow to be expected in a period of T years.

$Q\,(\text{Max.}) = Q\,(1 + 2\,A^{-0.3})$ in which Q (Max.) is the maximum rate of flow to be expected under the same conditions.

SUMMARY OF CONCLUSIONS.

(1) Though flood flows on different rivers vary greatly, some of the characteristics of the rivers affect the floods in substantially the same manner throughout the country; and expressions may be derived which show these effects.

(2) The effect of the size of the catchment area on the flood flows throughout the country is much the same; and this relation between the size of the catchment area and the size of the average yearly flood may be represented approximately by the expression,

$$Q\,(\text{Ave.}) = C\,A^{0.8}$$

such being true only in the case of the 24-hour average rate of flow of the floods.

(3) The relation between the maximum rate of flood flow on a stream in a period of years, and the maximum rate of flow for 24 hours during the same period, may be represented approximately by the expression,

$$Q\,(\text{Max.}) = Q\left(1 + \frac{2}{A^{0.3}}\right) = Q\,(1 + 2\,A^{-0.3}).$$

Table 1 gives the relative size for the maximum rate and the 24-hour average rate for different catchment areas, according to this relation.

(4) On streams throughout the country, floods which are a certain ratio of the average yearly flood occur with much the same frequency, and, on the average, the probable maximum flood in a period of years may be represented by the expression,

$$Q = Q\,(\text{Ave.})\,(1 + 0.8 \log. T).$$

TABLE 1.—RELATION BETWEEN MAXIMUM FLOOD AND AVERAGE 24-HOUR FLOOD.

$$Q\text{ (Max.)} = Q\ (1 + 2\ A^{-0.3}).$$

Catchment area, in square miles. (1)	Ratio of maximum flood to average 24-hour flood. (2)	Catchment area, in square miles. (1)	Ratio of maximum flood to average 24-hour flood. (2)
0.1	5.0	500	1.31
1.0	3.0	1 000	1.25
5.0	2.23	5 000	1.15
10	2.0	10 000	1.12
50	1.62	50 000	1.08
100	1.5	100 000	1.06

Table 2 gives the relative size of flood which may be expected in periods of years, according to the foregoing relation.

TABLE 2.—RELATION BETWEEN FLOOD TO BE EXPECTED IN A SERIES OF YEARS AND THE AVERAGE YEARLY FLOOD.

$$Q = Q\text{ (Ave.) } (1 + 0.8 \log. T).$$

Time, in years. (1)	Ratio of largest flood to average yearly flood. (2)	Time, in years. (1)	Ratio of largest flood to average yearly flood. (2)
1	1.00	50	2.36
5	1.56	100	2.60
10	1.80	500	3.16
25	2.12	1 000	3.40

(5) Coefficients may be obtained for streams by utilizing the foregoing relations to discount the effect of the length of period of observation and the size of the catchment area; these coefficients will serve as a gauge for the flood-producing capacity of the streams; and the difference in value of these coefficients is caused by the various physical characteristics of the river and its catchment area, such as storage, soil conditions, etc., and by the difference in the prevailing rainfall conditions.

Data.—The data available for a study on flood flows consist of records of floods on many streams covering periods of a comparatively few years, of a few continuous records for long periods, and of many scattered data of single great floods. In recent years, the U. S. Geological Survey has obtained observations on a large number of streams, either by its own observers or in conjunction with interested parties.

The Survey has also collected many data from old records. Many State commissions or officers have collected and published data. Some of the best records are those kept by water-power or manufacturing corporations.

Records of the same floods, as quoted in different reports and books, vary frequently. It has been possible sometimes to follow these records to their original source and obtain the most probable value, but undoubtedly many inaccuracies exist. It is believed, however, that in the main the data may be considered as sufficiently reliable to form a basis for a study of this character. At all events, it is the best available at the present time. It is very fortunate that there are a few records which are both accurate and complete, and cover periods of sufficient length to make them of great value. These observations have been made by those who appreciated the value of such records, and to whom accuracy was of the first importance.

The estimates that have been made of flow during great floods on rivers where regular gaugings have not been made are of considerable value as a check on the frequency of occurrence, although many of these estimates are only rough approximations, as an adequate basis for estimate is rarely at hand, especially after the event.

There are probably in existence many other data. The writer has used all that he has at hand, the accuracy of which was not questionable.

The method of rating rivers which is used in this paper, involving as it does simply the observation of the single largest flood in each year, makes it possible to rate a large number. An approximate value for the average flood, and for the coefficient, C, may be obtained from only 5 or 6 years' continuous records.

Effect of the Size of the Catchment Area.—The effect of the size of the catchment area on flood flows is first studied for 24-hour average floods. The method used is to compare the average yearly floods for different rivers. The use of these floods, though it eliminates the effect of the variable conditions involving time, does not in any way eliminate the effect of the constant factors, such as prevailing rainfall conditions, soil conditions, slope, storage, etc. These latter factors are of such importance that floods on streams in one section of the country may be many times larger than those on streams of similar size in another section; and, even in the same section, wide variation

may occur on streams of about the same size. In order to make a comparison which will show the effect of the size of the catchment areas, the streams must be grouped in such a way that the effect of these constant factors will be similar. There are so many of these factors that no grouping can be arranged that will make all of them even approximately equal. On the whole, it seems best to subdivide the country into sections which will have similar conditions as to rainfall and climate, and to rely on the average of a large number of streams to eliminate the effect of the remaining factors. For this purpose the following subdivision by the U. S. Geological Survey may serve as well as any other:

New England States,
Hudson River Basin,
Middle Atlantic States,
South Atlantic States,
Ohio River Basin.

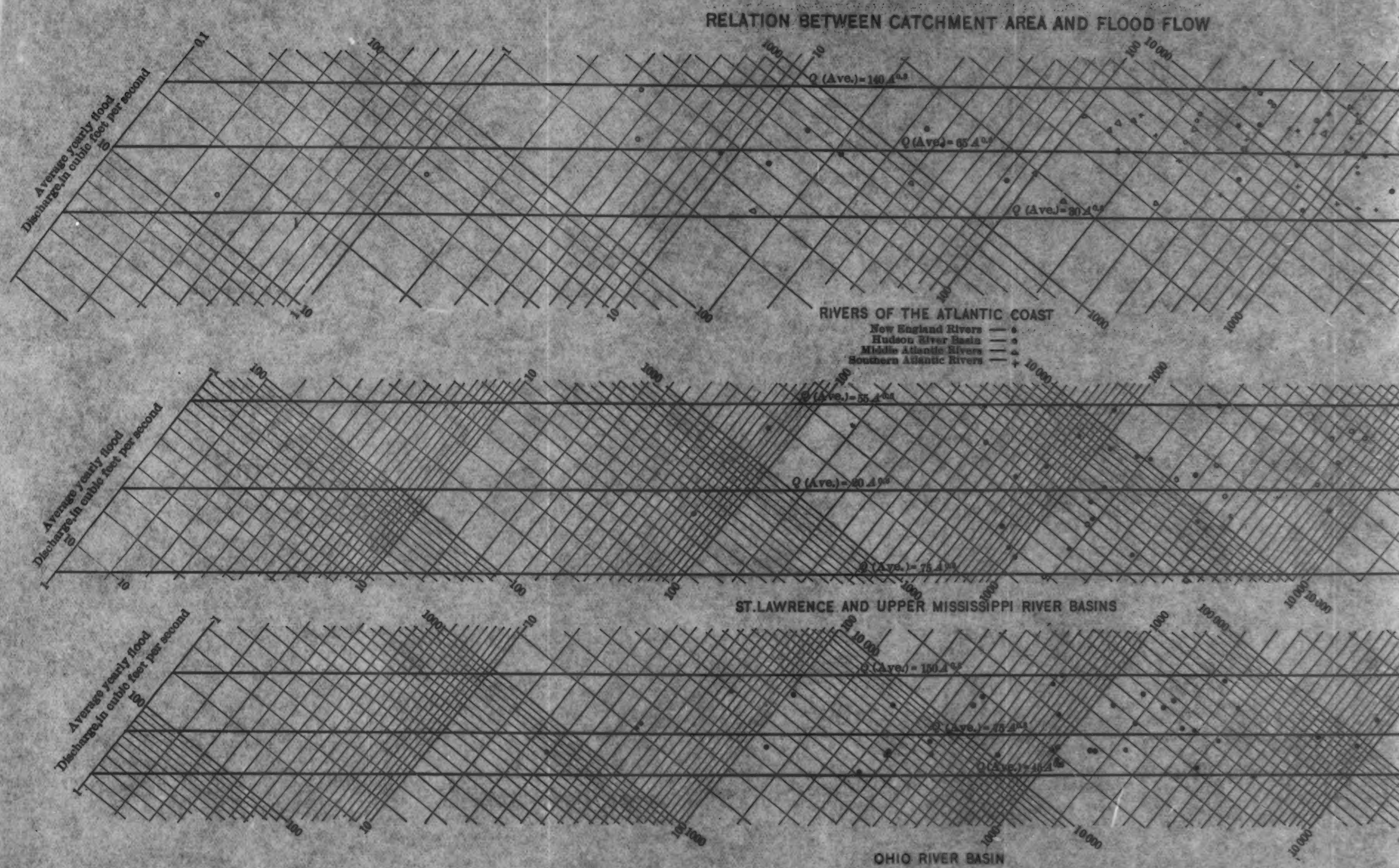
St. Lawrence River Basin,
Hudson Bay,
Upper Mississippi River Basin,
Misscuri River Basin,
Lower Mississippi River Basin.

Western Gulf of Mexico,
Colorado River Basin,
Great Basin,
Southern Pacific Coast,
Northern Pacific Coast.

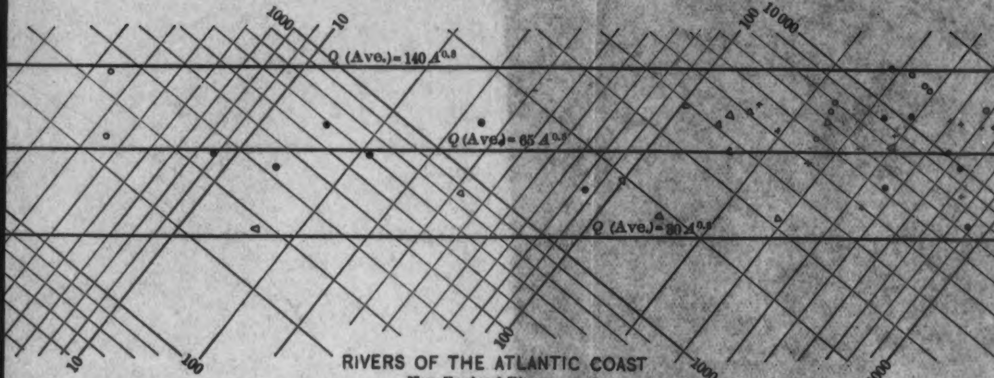
Tables 12 to 26 contain data on the average yearly floods for the rivers. The data from these tables are plotted on Plates IX, X, and XI.

The rivers in some of these divisions are so much alike in their characteristics that they have been plotted on the same diagram, as follows:

Plate IX. . . . . . {
New England States, Hudson River Basin, Middle Atlantic States, and South Atlantic States.
(*a*) Ohio River Basin.
(*b*) St. Lawrence River Basin and Upper Mississippi River Basin.

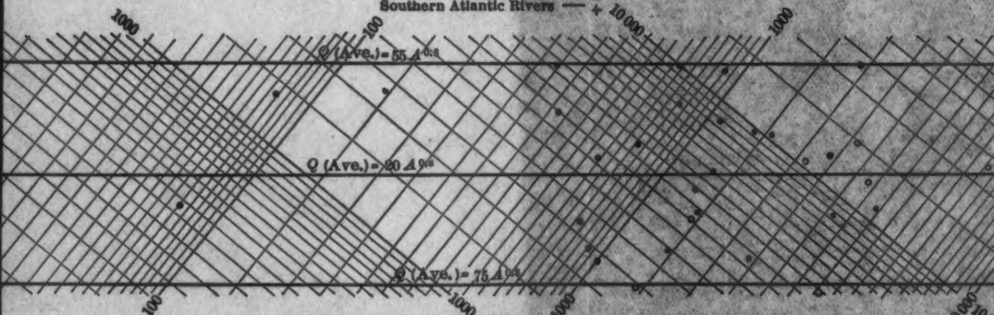

RELATION BETWEEN CATCHMENT AREA AND FLOOD FLOW
Average yearly flood
Discharge, in cubic feet per second
Q (Ave.) = 140 A^0.5
Q (Ave.) = 65 A^0.5
Q (Ave.) = 30 A^0.5
RIVERS OF THE ATLANTIC COAST
New England Rivers
Hudson River Basin
Middle Atlantic Rivers
Southern Atlantic Rivers
Q (Ave.) = 55 A^0.5
Q (Ave.) = 20 A^0.5
Q (Ave.) = 75 A^0.5
ST. LAWRENCE AND UPPER MISSISSIPPI RIVER BASINS
Q (Ave.) = 150 A^0.5
Q (Ave.) = 45 A^0.5
Q (Ave.) = 40 A^0.5
OHIO RIVER BASIN

RELATION BETWEEN CATCHMENT AREA AND FLOOD FLOW

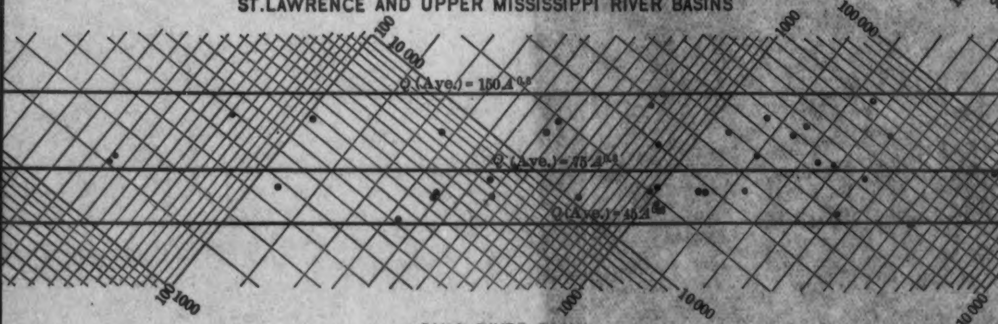


RIVERS OF THE ATLANTIC COAST

New England Rivers — •
Hudson River Basin — o
Middle Atlantic Rivers — △
Southern Atlantic Rivers — +



ST. LAWRENCE AND UPPER MISSISSIPPI RIVER BASINS



OHIO RIVER BASIN

PLATE IX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVII, No. 1293.
FULLER ON
FLOOD FLOWS.

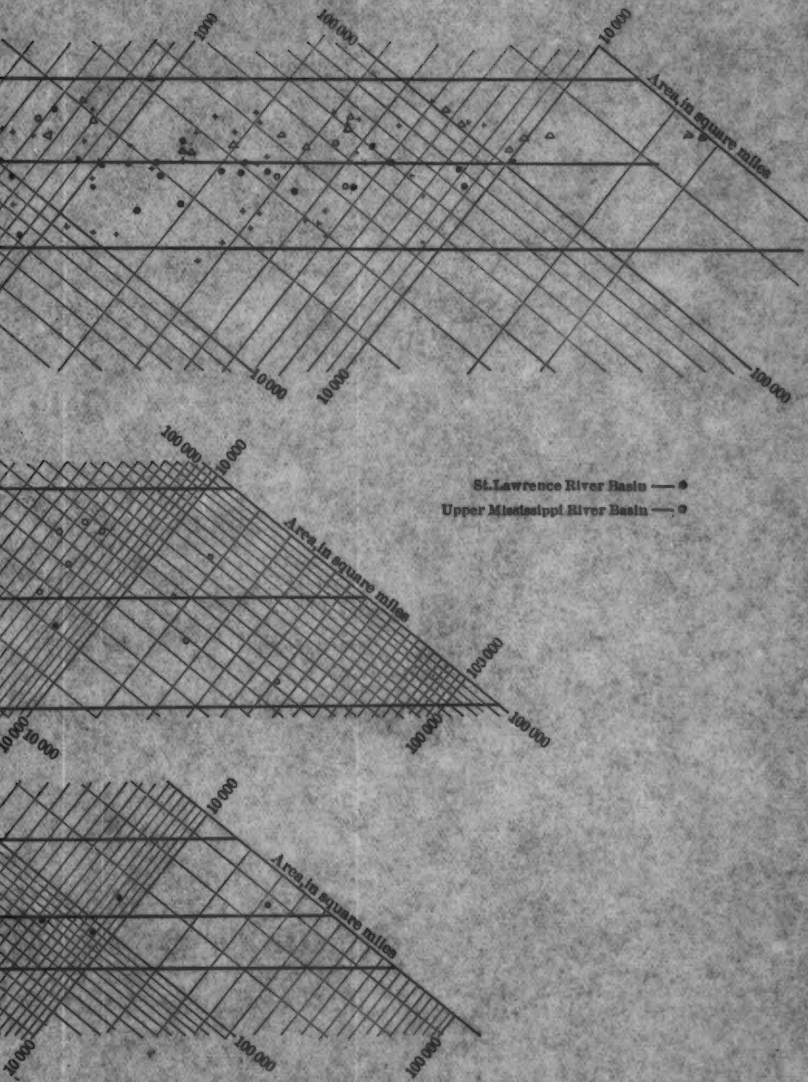


Plate X........	Missouri River Basin and Lower Mississippi River Basin. Western Gulf of Mexico and Colorado River Basin.
Plate XI........	Northern Pacific Coast and Southern Pacific Coast.

The rivers of the Great Basin and Hudson Bay vary so widely in their flood flows that no plottings have been made.

A study of these diagrams, which are on logarithmic scales, shows that, though the plottings cover zones of considerable width, the general slope of these zones is definite, and may be represented by the lines drawn. The data for the rivers on the Eastern Coast are the best, and the diagram for them shows the relation well. The slope of these lines is 0.8, so that it may be stated that the average rate of flood flow during 24 hours varies as the 0.8 power of the catchment area, or Q (Ave.) $= C\,A^{0.8}$.

The plotting for the Lower Mississippi and Missouri Rivers would indicate a slope of less than 0.8. An examination of the characteristics of the rivers on these basins, however, shows that the smaller ones are mainly mountain streams, and the larger ones flow through flat country. There is, therefore, an unbalanced element of difference in the data which may account for the apparent difference in the results.

The general indications do not justify any change in the value of the exponent for different sections; and though the data for some sections are not as satisfactory as for others, it is believed that this relation holds generally throughout the country.

All the plottings made on these diagrams are for a 24-hour average flow, and the formula, Q (Ave.) $= C\,A^{0.8}$, holds for such only. In this it differs from some other formulas proposed, and should not be compared with them.

Maximum Rate of Flood.—For all floods the maximum rate of flow will exceed the average for 24 hours. For the larger streams the rate of run-off will be of considerable magnitude for at least 24 hours, and for small streams a cloud-burst may cause a flood which will run off in a few hours, giving a large maximum rate with only a moderate 24-hour average rate. We should expect then that the maximum rate of flood, as compared with the average rate for 24 hours, will be greater for small than for large rivers.

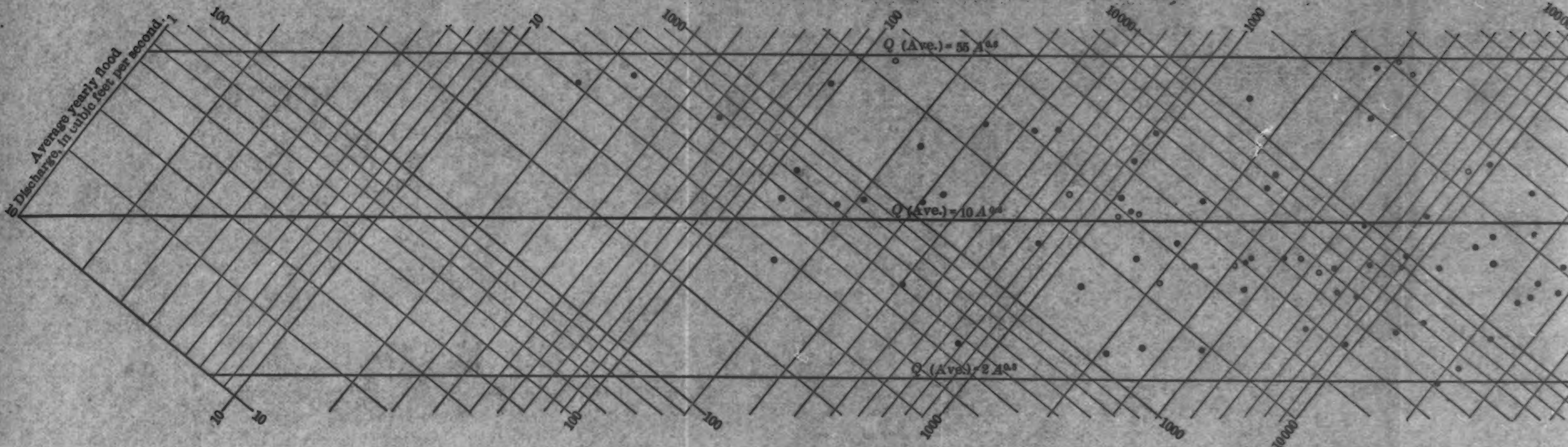
There are not sufficient data available to admit of a satisfactory independent study of the maximum rates in comparison with the sizes

of the catchment areas. It seems best, therefore, to study the relation between the maximum rate of flow and the 24-hour rate of flow. We are interested more particularly in the maximum rate of the great floods. That is, we want to ascertain the probable maximum rate of flood which is likely to occur in a long period. The greatest maximum rate of flood during a period of years does not necessarily occur during the same flood that gives the maximum 24-hour flood. The larger of the 24-hour floods are usually produced by storms of considerable duration, so that the relative excess of the maximum rate over the 24-hour average rate will not be so great for the large 24-hour floods as it will be for smaller 24-hour floods. In Table 3 is shown the rivers for which both the maximum and the average 24-hour rates were obtained. In some cases these represent different floods, but generally they repre-

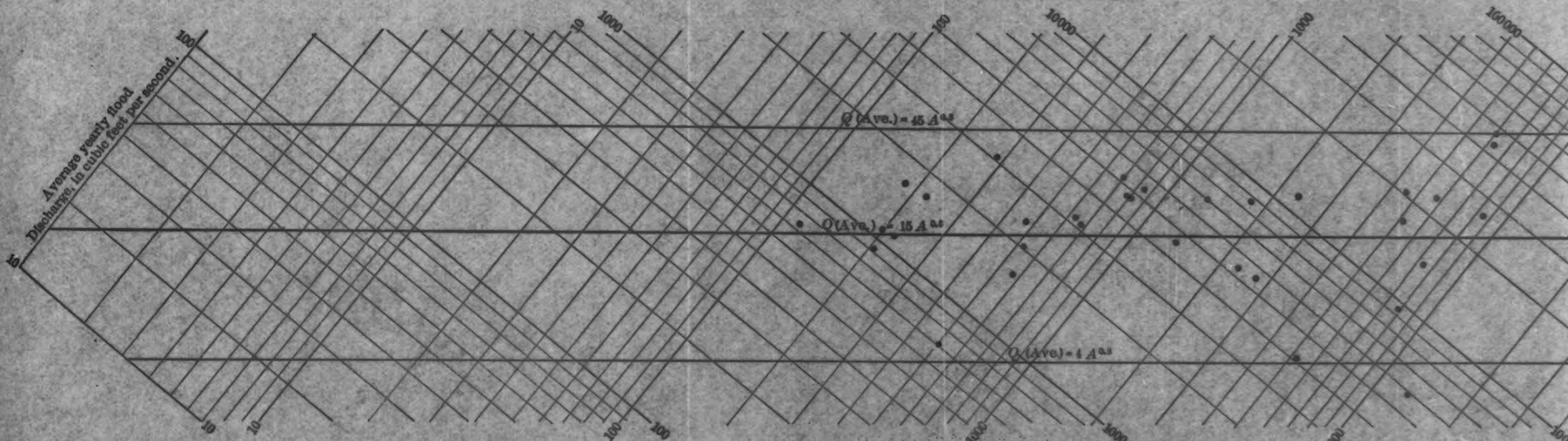
TABLE 3.—RELATION BETWEEN MAXIMUM RATE OF FLOW AND 24-HOUR AVERAGE RATE OF FLOW FOR CERTAIN FLOODS.

Name of river.	Catchment area, in square miles.	Rate of Flood Flow, in Cubic Feet per Second.		Ratio, maximum to average.		Ratio of excess of maximum over average to the average.
		Maximum.	Average for 24 hours.			
(1)	(2)	(3)	(4)	(5)		(6)
Sylvan Glen	1.18	337	87	3.87		2.87
		66.8	39.6	1.67		0.67
Starch Factory	3.4	372	250	1.48		1.48
		337	183	1.84		
Fomer (average of 4 largest floods)		1 123	434	2.60		
	13	2 379	788	3.00	3.08	2.08
		2 836	672	3.30		
		2 171	628	3.45		
Tohickon	102	14 100	8 650	1.63		0.63
Neshaminy	139	19 000	9 012	2.10		1.10
Oriskany	144	4 170	3 855	1.08		0.08
Perkiomen	152	17 600	8 769	2.01		1.01
Salmon	190.5	5 670	3 800	1.47		0.47
Piscataquis	286	22 200	18 100	1.22		0.22
Passaic	823	35 000	28 000	1.25	1.21	0.21
		25 000	21 400	1.18		
Genesee	1 070	42 000	33 170	1.27		0.27
Penobscot	1 100	25 700	21 400	1.20		0.20
Osage	1 237	38 500	33 900	1.14		0.14
Mohawk (average of 3 floods)		28 500	23 250	1.22		
	1 306	28 500	25 250	1.13	1.19	0.19
		27 220	22 560	1.20		
Belle Fourche	3 250	6 270	5 941	1.05		0.05
Yadkin	3 400	130 000	104 600	1.24		0.24
Kennebec	4 270	156 000	151 000	1.04		0.04
		156 800	127 000	1.23		0.23
Merrimac	4 638	90 000	74 000	1.22		0.22
Penobscot	6 600	96 700	76 300	1.26		0.26
Savannah	7 300	309 900	276 500	1.12		0.12
Allegheny	8 700	240 900	231 600	1.04		0.04
Susquehanna	24 030	700 000	590 000	1.19		0.19
Susquehanna (ice dam)	26 766	691 000	352 900	1.90		0.90
Kansas	58 530	228 500	221 000	1.04		0.04

RELATION BETWEEN CATCHMENT AREA AND FLOOD FLOW

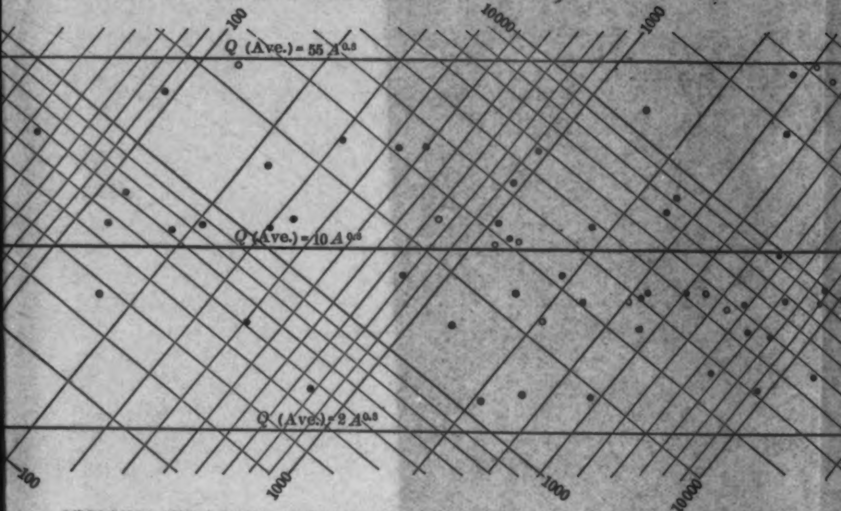


MISSOURI AND LOWER MISSISSIPPI RIVER BASINS

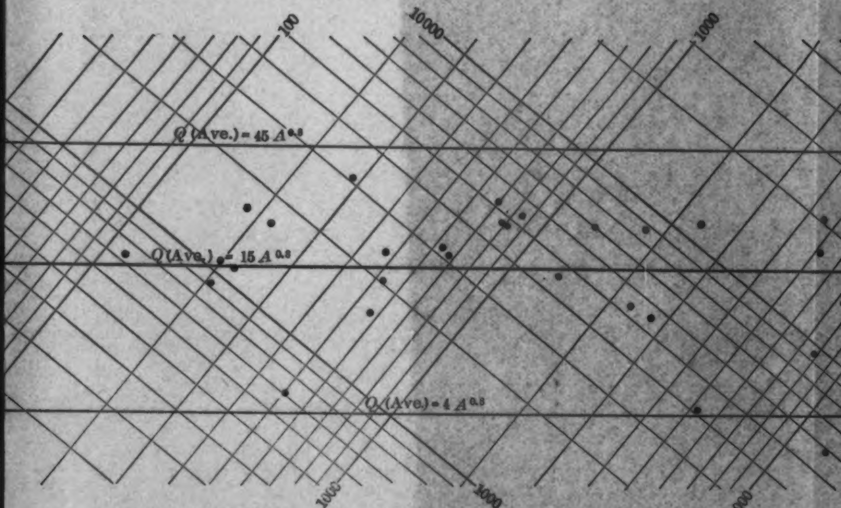


COLORADO RIVER BASIN

ON BETWEEN CATCHMENT AREA AND FLOOD FLOW



MISSOURI AND LOWER MISSISSIPPI RIVER BASINS



COLORADO RIVER BASIN

PLATE X.
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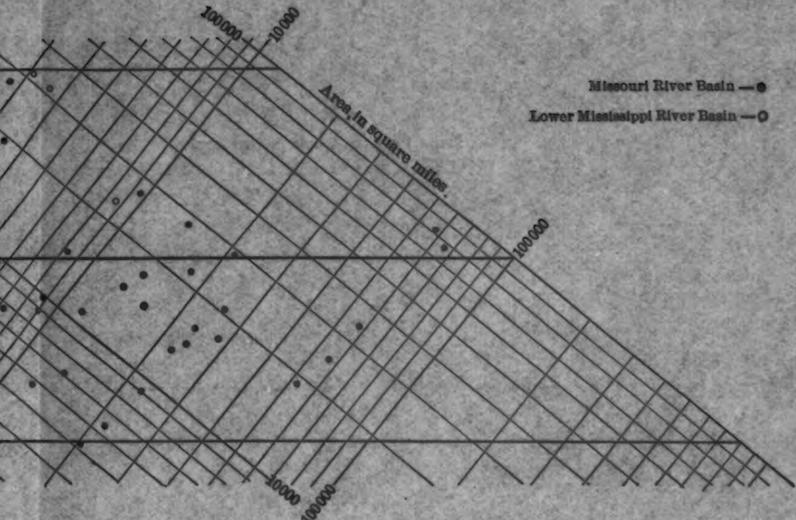
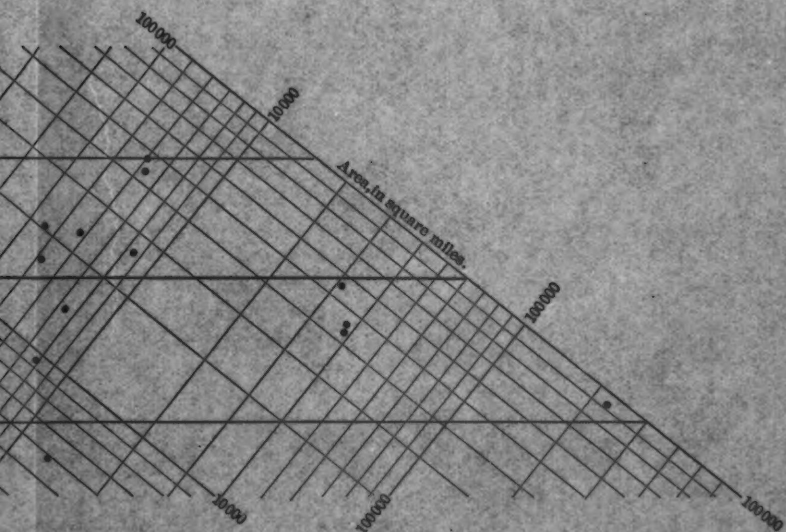
FULLER ON
FLOOD FLOWS.

Missouri River Basin —●

Lower Mississippi River Basin —○





sent the same one. These are mainly large floods, but do not necessarily represent the maximum flood on the river. It is probable that the relation, as obtained by the average of the data in Table 3, represents one large enough for use in obtaining the probable maximum rate to be expected in a considerable term of years, although some smaller floods may have a much greater excess of maximum rate over the 24-hour average rate.

The method of obtaining the relation between the two rates of flow is shown on Table 3 and on Plate XII. The ratio of the excess of the maximum rate of flood over the 24-hour average rate to the average rate is plotted to a logarithmic scale in relation to the catchment area, and an average curve is drawn. From this curve may be obtained the relation:

$$\frac{Q\text{ (Max.)} - Q}{Q} = 2\,A^{-0.3} \text{ or } Q\text{ (Max.)} = Q\,(1 + 2\,A^{-0.3}).$$

This should be considered only as an approximate average relation, from which considerable deviations in particular cases are to be expected.

The great rate of flood on the Susquehanna which is shown on the diagram was due to an ice jam, and may be considered as a rare occurrence. It should perhaps be considered as a matter of frequency of flood flows rather than as a relation between the maximum rate and the 24-hour average rate. That is, such an occurrence comes probably only in a long period of time, and during that period larger 24-hour average floods than the one given for this flood would normally occur, so that the relation between the two rates in a considerable period of years would be less than is indicated by this particular flood. Frequently, the ice dam, though it would cause a great flood immediately below the dam, would not increase to a like extent the flood some distance farther down stream, as the temporary storage along the river and in ponds and lakes will take up the water let loose by it and equalize the flow.

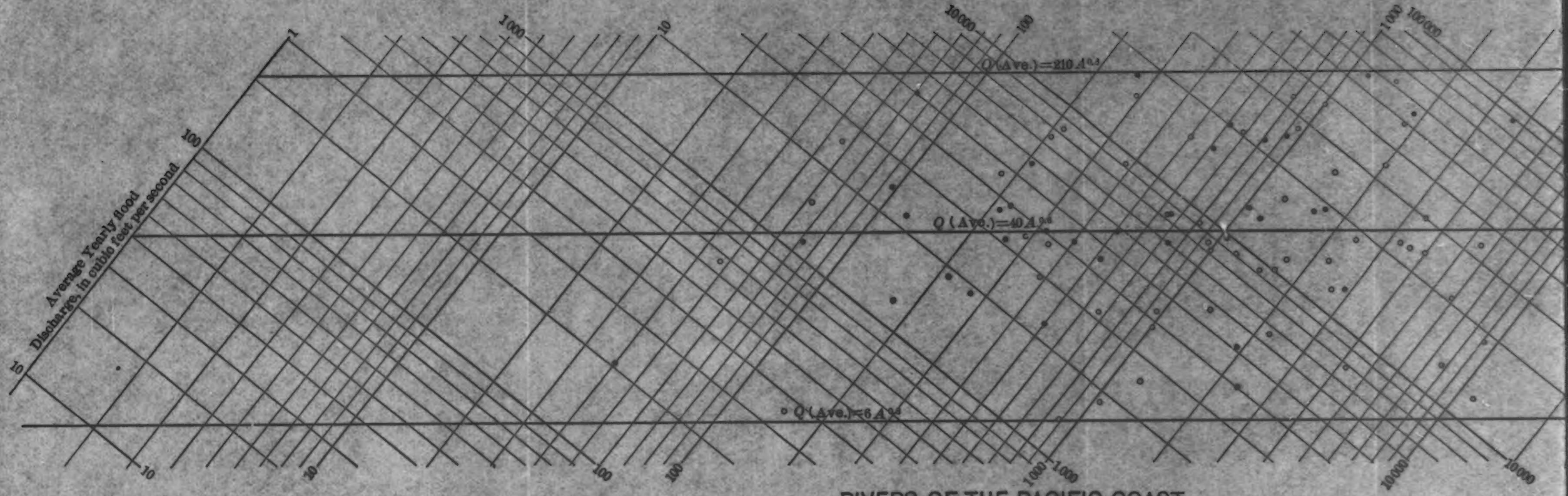
Frequency of Occurrence of Floods.—If the data for all the floods that have occurred in a single river for several hundred years were available, a relation could be established showing the average frequency with which floods of any size occur. As such data are not available, it becomes necessary, in order to establish the frequency relation, to utilize data for shorter periods on many rivers. Special

studies were made to see if this relation was similar for different rivers. Although considerable variation occurred for the shorter-term records, these studies showed clearly that a general law could be established. In Tables 12 to 26, the larger floods on many different rivers are compared by using the proposed formula for frequency. An inspection of these tables shows that the values of the coefficient thus obtained are, in most cases, reasonably constant, and are similar to the values obtained by comparison with the average yearly flood, thus indicating that the frequency relation holds for the different rivers.

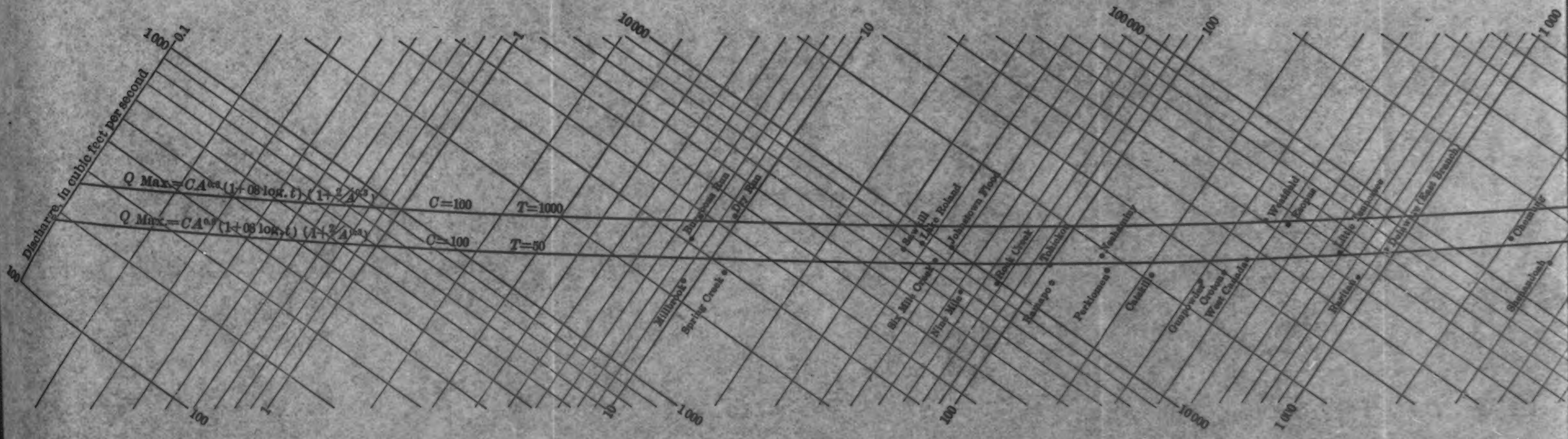
As rivers follow the same general law, it is allowable to use the data on all the rivers in the same way as those on a single river, provided such data can be put on a common basis. The use of the yearly average flood affords a means of doing this. If the same law holds, the ratio of the larger floods to the yearly average floods should be the same for all rivers for the same period of time. The method of comparing the data and obtaining the frequency is explained in detail later.

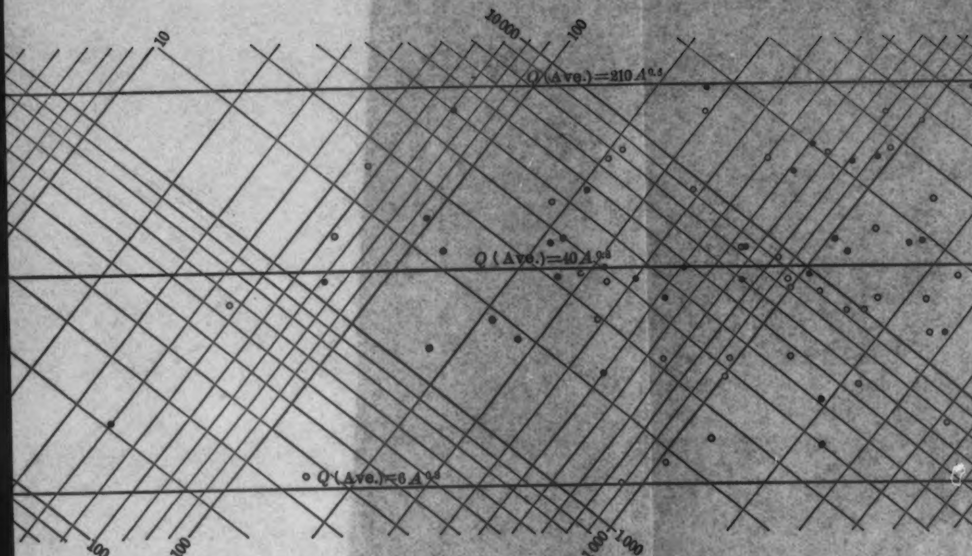
As most of the data were obtained during the last 10 or 20 years, objections to this method of analysis may be raised on the score that weather conditions may have been different during other periods in the past, or may be different in the future. Certainly, if weather conditions throughout the country were subject to permanent changes, this objection would be well taken. There is no evidence, however, that this is so. The rivers considered are distributed throughout the United States, so that local conditions should not affect the results. We know that a dry season in one section of the country may occur during the same year as a wet season in another section. We also know that storms which cause great floods on small streams may cause only moderate floods on larger streams, and *vice versa.* Considering these facts, the diversity of the conditions, and the large area over which the rivers are distributed, it seems that periods of 10 years or more give a fair average of the conditions which have existed in the past and will probably exist in the future.

As a check on this frequency relation we have long-term records for a few rivers, which show that the relation holds approximately for these rivers. A few of these records cover a century. A further check is given by the lower diagram on Plate XI, on which the greatest floods of the century are plotted, showing a close relation to the proposed formula.



RIVERS OF THE PACIFIC COAST





RIVERS OF THE PACIFIC COAST

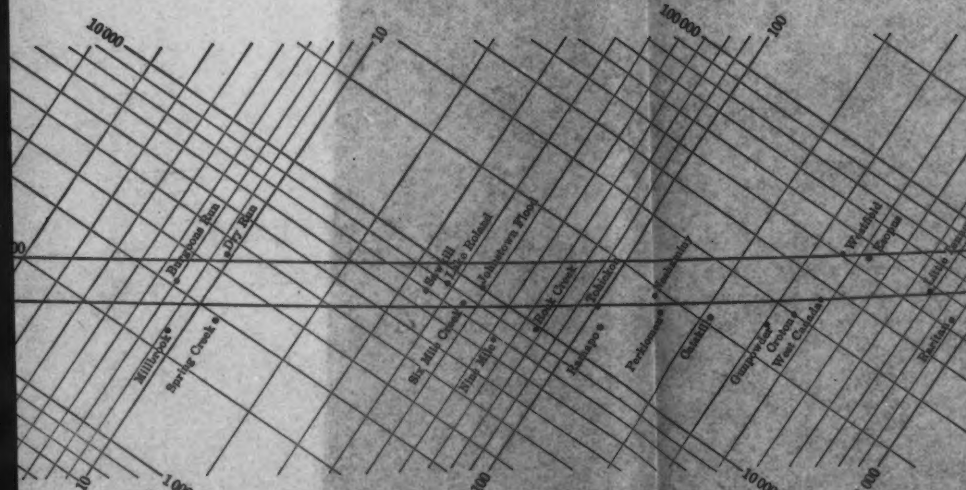


PLATE XI.
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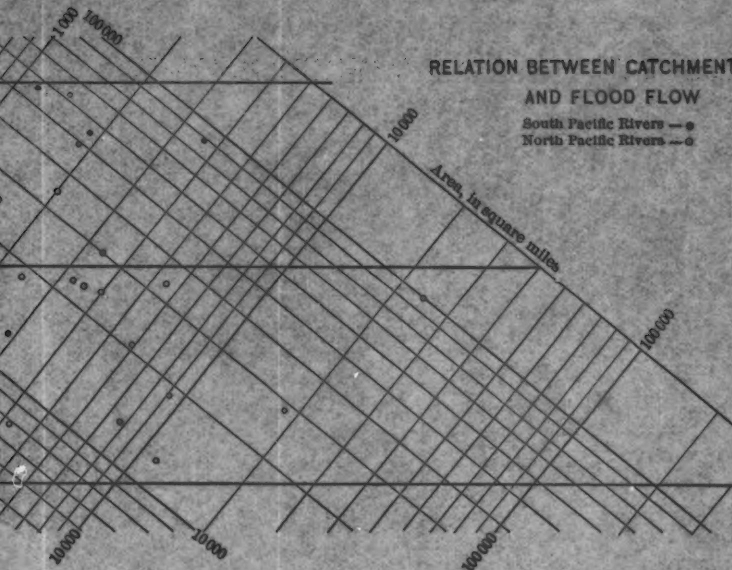
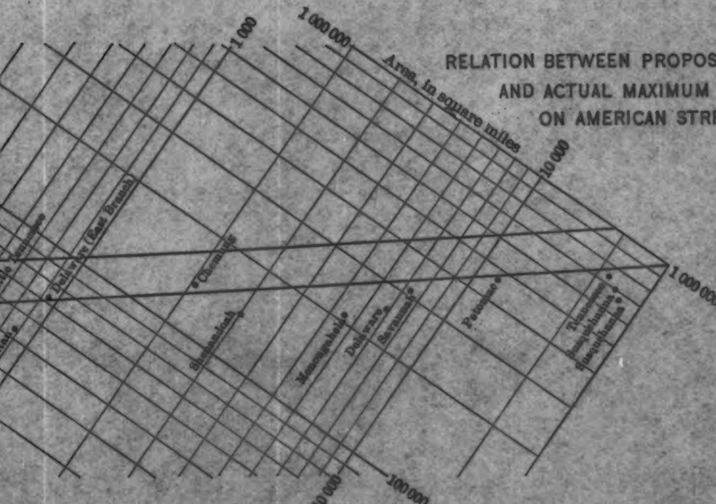
RELATION BETWEEN CATCHMENT AREA AND FLOOD FLOW

South Pacific Rivers — ●
North Pacific Rivers — o



RELATION BETWEEN PROPOSED FORMULA AND ACTUAL MAXIMUM FLOODS ON AMERICAN STREAMS



Method of Determining the Frequency.—As there is some choice as to the flood to be selected in the study for frequency, some matters involving the general method which has been followed are here given.

If we are given a record of floods for a series of years, we may divide it into a number of equal periods of time. Taken independently, the maximum flood in each period might be said to represent the probable flood to be obtained in that period. These maximum floods, however, will vary, and we may select the most probable maximum flood which will occur in a given period by any of the following methods.

First, the average of these maximum floods, which may be called the average maximum flood to be expected in this period of time; second, the one of these maximum floods for which there is one greater for each one less, which may be called the median maximum flood to be expected in the period of time; third, the smallest of these maximum floods, which may be considered as the flood that should be equalled or exceeded in the period of time.

As the yearly floods considered range from a fraction of the average to several times this average, it may be readily seen that there will probably be more floods of less than, than there will be of more than, the average. The average of the maximum floods, then, will exceed the median flood. It is a question which of these two floods better represents the requirements. The average flood is more easily obtained and is more accurate, particularly for short-term records. It is larger, thus leading to safe design. As the study is based on the average yearly floods, it seems better to continue the method and use the average maximum flood. The relation between the foregoing different floods seems to be constant, as is shown in the detailed method given later. It may be stated that it would have been possible to have used the median flood from the start, to have selected the median yearly flood, and to have made the study on that basis. It is believed, however, that, on the whole, the average flood method is the better.

It should be noted that the results are also affected by two other matters. First, only the single largest flood in each year is used. This eliminates all the others during that year, although they may have been greater than the largest flood in other years. If these second and third floods in some years were included in the study, they would tend to give somewhat higher values for the probable flood. On the

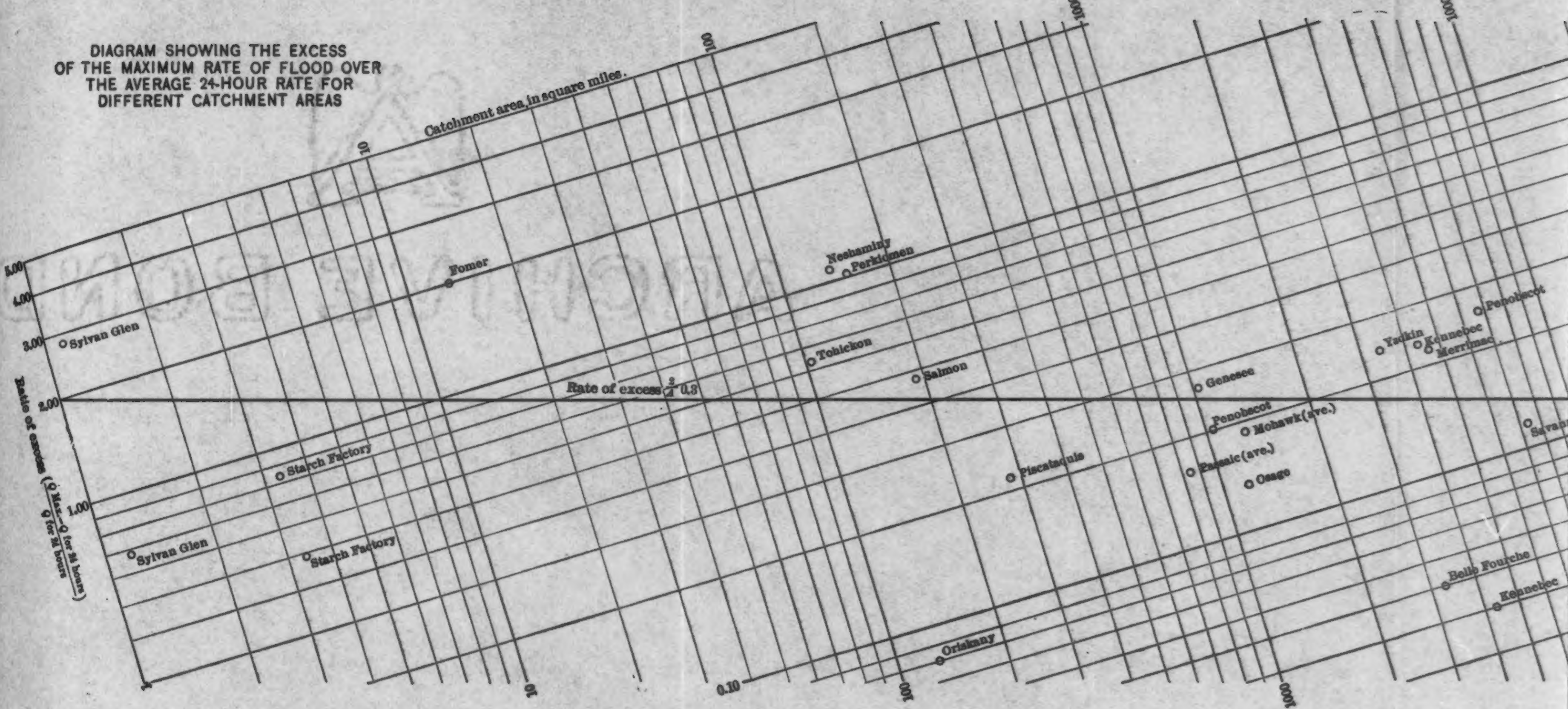
other hand, in the method of analysis no attention is paid to the fact that some of the largest floods may come close together. It is assumed that the floods are uniformly distributed over the period, though, in fact, some periods would have two or more floods of the size under consideration while others would not contain any such flood. If this were taken into account, it would tend to reduce somewhat the value of the probable flood to be expected. These errors in methods, if they may be so called, are not of great moment. Generally, it may be said that the flood obtained by the proposed formula is probably somewhat in excess of that which we would expect to get in any one period, but it is the average of the floods in a number of periods of the same length.

General Method.—The general methods of treating the data can be best shown by a specific example. For this purpose, the writer has used Tohickon Creek, the data for which are contained in the reports of the Philadelphia Water Board for 25 years. All the steps leading up to the determination of the average flood and the frequency of occurrence of floods on this stream are shown. In working out the general formula for frequency for all rivers, the same method is followed as here given, the data for all the rivers being collected and used as if they were for one stream. This example then illustrates the entire method of procedure. The first step is to select from the daily records the largest single flood for each year. These are shown in Column 2 of Table 4. The second step is to average all these floods, as is shown at the foot of Column 2. The third step is to select and arrange the floods in order of their magnitude. This is indicated in Table 4, Q_1 representing the largest flood; Q_2 the second; Q_3 the third, etc. The next step is to find the ratio of these larger floods to the average flood. This is shown in Column 3. The next step is to arrange the data so that the frequency may be determined. The methods for doing this are shown in Table 5.

The methods of determining all the three different values for the probable flood to be expected, as previously stated, are here given.

Method of Obtaining Average Maximum Flood to be Expected.—The method of computing the average maximum flood to be expected is shown in Table 5, Part *a*. Column 1 gives the serial number of each flood, in order of size. Column 2 gives the ratio of the flood to the average flood, as taken from Table 4. Column 3 gives the summation

DIAGRAM SHOWING THE EXCESS
OF THE MAXIMUM RATE OF FLOOD OVER
THE AVERAGE 24-HOUR RATE FOR
DIFFERENT CATCHMENT AREAS



100
1000
Neshaminy
Perkiomen
Tohickon
Salmon
Genesee
Yadkin
0.3
Penobscot
Mohawk (ave.)
Passaic (ave.)
Piscataquis
Osage
Oriskany
0.10
100
1000

PLATE XII.
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of all the floods equal to or exceeding the flood concerned. Column 4 gives the summation in Column 3 divided by the serial number. Column 5 gives the total number of years in the record divided by the serial number. It is evident, then, that Column 4 represents the average of the floods which occur in the period of years given in Column 5. On the first diagram of Fig. 1 the values in Columns 4 and 5 are plotted on a logarithmic scale for the values of the time (Column 5) and on an ordinary scale for the values of the ratios (Column 4); these plottings are marked *A* on the first diagram of Fig. 1.

TABLE 4.—METHOD OF TREATING THE RECORDS TO OBTAIN THE AVERAGE YEARLY FLOOD Q AVERAGE, AND TO OBTAIN THE RATIO OF MAXIMUM FLOODS.

River, Tohickon ; Station, Pt. Pleasant, Pa.; Catchment Area, 102 sq. miles. $A^{0.8} = 58.5$.

Date. (1)	Maximum 24-hour average flood for the year. (2)	Ratio of flood to average yearly flood. (3)
1884	4 379 Q_8	1.06
1885	3 664	....
1886	5 359 Q_4	1.30
1887	2 544	....
1888	3 493	....
1889	4 714 Q_5	1.15
1890	2 942	....
1891	2 858	....
1892	3 158	....
1893	2 994	....
1894	8 650 Q_1	2.10
1895	3 857	....
1896	6 515 Q_2	1.59
1897	3 688	....
1898	4 160 Q_{10}	1.01
1899	3 222	....
1900	No record.	....
1901	4 089 Q_{12}	0.99
1902	5 958 Q_3	1.45
1903	4 968 Q_6	1.21
1904	4 395 Q_7	1.06
1905	4 175 Q_9	1.01
1906	3 200	....
1907	4 120 Q_{11}	1.00
1908	2 770	....
1909	3 050	....

Average yearly flood Q (Ave.) $102\,917 \div 25 = 4\,117$.

Median Flood.—The method of obtaining the median maximum flood is shown in Table 5, Part *b*. The second largest flood is here taken as representing the size of that one which will probably come in one-third of the total period. That is, there will be one flood greater, one less, and one equal to it in the three periods. In the

same way, the third flood is taken as the most probable one for a period equal to one-fifth of the total time, and so on. Column 1 gives the serial number from Table 4. Column 2 gives the corresponding ratio; Column 3 gives the time for which this flood is the median, and is obtained by dividing the total time by one less than twice the serial number. Columns 2 and 3 are then plotted as for the first method. These plottings are marked *B* on the first diagram of Fig. 1.

TABLE 5.—TOHICKON CREEK.

PART *a*.—METHOD OF OBTAINING PROBABLE AVERAGE MAXIMUM FLOOD TO BE EXPECTED.

No. of flood, in order of magnitude. (1)	Ratio of flood to average yearly flood. (2)	Summation of ratios. (3)	Summation of ratios ÷ No. of floods. (4)	Time, in years. (5)
1..........	2.10	2.10	2.10	25.00
2..........	1.59	3.69	1.85	12.50
3..........	1.45	5.14	1.71	8.33
4..........	1.30	6.44	1.61	6.25
5..........	1.21	7.65	1.53	5.00
6..........	1.15	8.80	1.47	4.17
7..........	1.06	9.86	1.41	3.57
8..........	1.06	10.92	1.36	3.33
9..........	1.01	11.93	1.33	2.79
10..........	1.01	12.94	1.29	2.50
11..........	1.00	13.95	1.27	2.27

PART *b*.—METHOD OF OBTAINING MEDIAN MAXIMUM FLOOD.

No. of flood. (1)	Ratio. (2)	Time, in years. (3)
2..............	1.59	8.33
3..............	1.45	5.00
4..............	1.30	3.57
5..............	1.21	2.79
6..............	1.15	2.27

PART *c*.—METHOD OF OBTAINING PROBABLE FLOOD THAT WILL BE EQUALLED OR EXCEEDED DURING A PERIOD.

No. of flood. (1)	Ratio. (2)	Time, in years. (3)
2..............	1.59	12.50
3..............	1.45	8.33
4..............	1.30	6.25
5..............	1.21	5.00
6..............	1.15	4.17
7..............	1.06	3.57
8..............	1.06	3.33
9..............	1.01	2.79
10..............	1.01	2.50
11..............	1.00	2.27

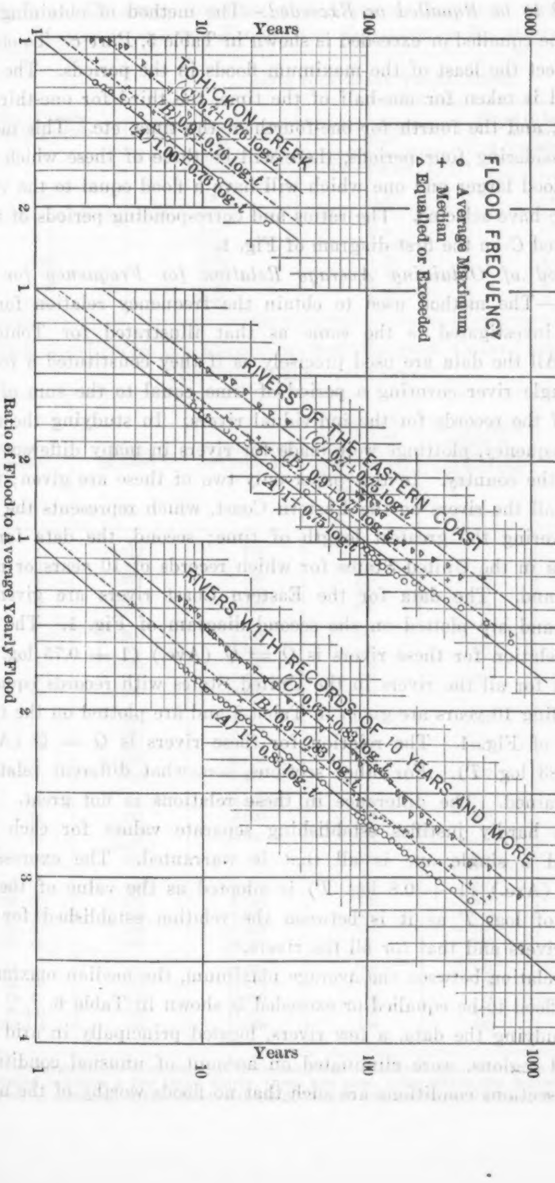

FLOOD FREQUENCY
o Average Maximum
+ Median
△ Equalled or Exceeded
TOHICKON CREEK
(C) 0.7+0.76 log T
(B) 0.9+0.78 log T
(A) 1.00+0.78 log T
RIVERS OF THE EASTERN COAST
(C) 0.7+0.75 log T
(B) 0.9+0.75 log T
(A) 1+0.75 log T
RIVERS WITH RECORDS OF 10 YEARS AND MORE
(C) 0.6+0.83 log T
(B) 0.9+0.83 log T
(A) 1+0.83 log T
Years
Years
1
10
100
1000
1
2
1
2
1
2
3
Ratio of Flood to Average Yearly Flood

FIG. 1

Flood to be Equalled or Exceeded.—The method of obtaining the flood to be equalled or exceeded is shown in Table 5, Part *c*. To obtain it we select the least of the maximum floods in the periods. The second flood is taken for one-half of the time, the third for one-third of the time, and the fourth for one-fourth of the time, etc. This means that, considering four periods, there will be three of these which will have a flood larger and one which will have a flood equal to the value which we have selected. The ratios and corresponding periods of time are marked *C* on the first diagram of Fig. 1.

Method of Obtaining Average Relation for Frequency for All Streams.—The method used to obtain the frequency relation for all streams investigated is the same as that illustrated for Tohickon Creek. All the data are used precisely as if they constituted a record for a single river covering a period of time equal to the sum of the length of the records for the individual rivers. In studying the matter of frequency, plottings were made for rivers in many different sections of the country. In this paper only two of these are given; first, that for all the rivers on the Eastern Coast, which represents the best data, covering the greatest length of time; second, the data for all the rivers in the United States for which records of 10 years or more are at hand. The data for the Eastern Coast rivers are given in Table 7 and are plotted on the second diagram of Fig. 1. The frequency relation for these rivers is $Q = Q$ (Ave.) $(1 + 0.75 \log. T)$. The data for all the rivers in the United States with records equal to or exceeding 10 years are given in Table 8 and are plotted on the third diagram of Fig. 1. The relation for these rivers is $Q = Q$ (Ave.) $(1 + 0.83 \log. T)$. For other sections, somewhat different relations were obtained. The difference in these relations is not great. The variation hardly justifies establishing separate values for each section, and a single one is all that is warranted. The expression, $Q = Q$ (Ave.) $(1 + 0.8 \log. T)$ is adopted as the value of the coefficient of log. T as it is between the relation established for the eastern rivers and that for all the rivers.

The relation between the average maximum, the median maximum, and the flood to be equalled or exceeded is shown in Table 6.

In studying the data, a few rivers, located principally in arid and semi-arid regions, were eliminated on account of unusual conditions. In such sections conditions are such that no floods worthy of the name

TABLE 6.—RELATION BETWEEN THE AVERAGE MAXIMUM, THE MEDIAN MAXIMUM, AND THE FLOOD TO BE EQUALLED OR EXCEEDED.

No. of years in the period.	Ratio to Average Yearly Flood.		
	Average maximum flood $Q = Q$ (Ave.) $(1 + 0.8 \log. T)$.	Median maximum flood Q (median) $= Q$ (Ave.) $(0.9 + 0.8 \log. T)$.	Flood to be equalled or exceeded Q (equalled or exceeded) $= Q$ (Ave.) $(0.7 + 0.8 \log. T)$.
(1)	(2)	(3)	(4)
1..........	1.0	0.9	0.7
5..........	1.56	1.46	1.26
10..........	1.80	1.70	1.50
50..........	2.36	2.26	2.06
100..........	2.60	2.50	2.30
1 000..........	3.40	3.30	3.10

TABLE 7.—FLOODS ON RIVERS OF THE EASTERN COAST, FOR WHICH RECORDS WERE AVAILABLE.

Serial number.	Ratio of flood.	Summation of Column 2.	Column 3 divided by Column 1; average of flood in this period.	Median flood which is exceeded in one-half the periods.	No. of years in the period.
(1)	(2)	(3)	(4)	(5)	(6)
1	312	312	312	...	1 532
2	305	617	308	...	766
3	271	888	296	305	511
4	271	1 159	289	...	384
5	264	1 423	285	271	306
6	260	1 683	280	...	255
7	257	1 940	277	271	219
8	255	2 195	274	...	192
9	250	2 445	272	264	17C
10	250	2 695	270	...	158
11	248	2 942	267	260	139
12	234	3 177	264	...	128
13	231	3 408	262	257	118
14	226	3 634	259	...	110
* 13	225	3 378	260	257	116
14	221	3 599	257	...	108
15	221	3 820	254	255	100
16	{221 / 210}	4 041	252	...	94
21	205	4 930	234	228	64
22- 31	186	6 846	220	210	44
32- 38	177	8 066	212	205	34
39- 49	172	9 981	203	193	27
50- 54	167	10 564	196	188	23
55- 63	160	12 021	191	186	19
64- 68	154	12 689	187	185	16
69- 78	151	14 214	182	176	14
79- 89	156	15 680	176	174	11
90-100	144	17 281	173	171	10
101-114	141	19 278	168	163	9

* Two large floods eliminated, as the record was not continuous. For such cases, where the record includes a few large floods, these are used, after which the computations are adjusted by eliminating the effect of these floods and proceeding as if they had not existed.

occur in some years, though occasionally there is a large one. For such cases the average yearly flood becomes low and the percentage of the maximum flood becomes unduly high. These conditions are quite different from those generally found, and it seems best to exclude such cases. It follows, of course, that the frequency relation here proposed does not hold for such rivers.

TABLE 8.—FLOODS ON ALL RIVERS FOR WHICH RECORDS OF TEN YEARS OR MORE WERE AVAILABLE.

Serial number. (1)	Ratio of flood. (2)	Summation of Column 2. (3)	Column 3 divided by Column 1; average of flood in this period. (4)	Median flood which is exceeded in one-half the periods. (5)	Number of years in the period. (6)
1.....	373	...	373	...	1 672
2.....	312	685	342	...	836
3.....	293	978	326	312	557
4.....	291	1 269	315	...	418
5.....	276	1 545	309	293	337
6.....	270	1 815	302	...	278
7.....	267	2 082	297	291	239
8.....	264	2 346	293	...	209
9.....	262	2 608	289	276	186
10.....	253	2 861	286	...	167
11.....	250	3 111	282	270	152
12.....	242	3 353	279	...	139
13.....	238	3 591	276	267	129
14.....	236	3 827	273	...	120
15.....	235	4 062	270	264	111
16.....	234	4 296	268	...	104
17.....	231	4 527	266	262	99
18.....	231	4 758	264	...	93
19.....	226	4 084	262	253	88
20.....	224	5 208	260	...	84
20- 30.....	210	7 352	244	235	56
30- 40.....	200	9 410	235	224	42
40- 50.....	193	11 376	227	...	34
50- 60.....	185	13 250	221	210	28
60- 70.....	181	15 073	215	...	24
70- 80.....	175	16 844	210	200	21
80- 90.....	172	18 578	206	...	19
90-100.....	169	20 286	203	193	17
100-110.....	167	21 961	199	...	15
110-120.....	162	23 603	196	185	14
120-130.....	158	25 197	193	...	13
130-140.....	156	26 768	191	181	12
140-150.....	153	28 310	189	...	11
150-160.....	150	29 825	187	175	10.5
160-170.....	148	31 318	185	...	10
170-180.....	144	32 769	182	172	9.5
180-190.....	141	34 190	180	...	9.0
190-200.....	139	35 587	178	169	8.5
200-210.....	136	36 461	176	...	8.0
210-220.....	134	38 319	174	167	7.5
....	...	......	...	134	3.75

Table 9 contains data on a few of the largest relative floods in the United States, as compared with the average yearly floods.

TABLE 9.—SOME OF THE LARGER FLOODS, AS COMPARED WITH THE YEARLY AVERAGE FLOODS.

No.	River.	Drainage area, in square miles.	Point of observation.	Floods, in Cubic Feet per Second. Maximum.	Average.	Ratio of maximum to average.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1....	Kansas........	58 550 59 841	Compton Lawrence } Kans.	221 000	59 000	3.73
2....	Shenandoah...	3 000	Millvale, W. Va.......	139 700	44 800	3.12
3....	South Platte...	633	Denver, Colo..........	5 570	1 900	2.93
4....	Truckee.......	955	State Line, Cal........	15 300	5 260	2.91
5....	Tuolumne......	1 500	La Grange, Cal........	52 000	18 900	2.76
6....	Savannah......	7 300	Augusta, Ga...........	310 000	114 300	2.70
7....	Schuylkill.....	1 920	Philadelphia, Pa.......	82 156	30 700	2.67
8....	Passaic........	823	Dundee, N. J..........	27 995	10 600	2.64
9....	Umatilla.......	353	Gibbon, Ore...........	10 000	3 808	2.62
10..	Kennebec......	4 270	Waterville, Me.........	151 000	59 600	2.53

USE OF THE FORMULA.

$$Q = C\,A^{0.8}\,(1 + 0.8 \log. T).$$

$$Q\,(\text{Max.}) = Q\,(1 + 2\,A^{-0.3}).$$

Selecting the Value for the Coefficient, C.—Tables 12 to 26 contain values for the coefficient, *C*, for many rivers in the United States. These coefficients are applicable for the river at the station where the floods were observed and for other points on the river where conditions are not materially different. Obviously, the coefficient for a river above large branches or above points where there may be considerable storage may be different from that of the river below. Slopes and other conditions affect the coefficient, and for the river at other points than where the gaugings were made, judgment must be used in its selection. The coefficient for the upper portion of a river may be different from that for the lower portion. The coefficient for the river, however, taken in connection with others for similar rivers, should enable one to select an approximate value for any river in the tables. For streams for which no coefficients are given in the tables, approximate values may be obtained by comparison with rivers having similar characteristics, for which the coefficients are given.

It may be possible to obtain local data as to the height of the ordinary yearly flood, which may give an indication of the value of the coefficient. It may also be possible to obtain records of one or two of the larger floods on the stream, and if one can ascertain the time

when these occurred, the data may be reduced by the formula of frequency to obtain approximate values of C.

Some data of this latter character are contained in Table 28. No attempt has been made to reduce these data to find the value of C, as the information in regard to the period represented is not definite. Local investigation of the conditions might enable one to form a reasonable estimate as to the period of time these floods represent. It is probable that records of other floods on these streams would be available, together with the time of the occurrence, so that approximate values for the coefficients could be established.

The effect on flood flow of such factors as slope of drainage basin, shape of drainage basin, rate of rainfall, etc., have been studied and discussed in a number of papers. For the sake of convenience, and to aid in the selection of the coefficient, C, from the tables, it may be well to summarize briefly the effect of some of these factors, stating generally their effect on the magnitude of the flood.

Storage.—Storage may be in a reservoir, or in the ground. Generally speaking, all storage tends to reduce the size of floods. Rivers on which there are many lakes or reservoirs, or on which there are large sandy areas, generally have low flood flows, as compared with rivers otherwise similar. When the lakes and reservoirs are full, the storage below the overflow is not available for reducing floods. Storage above the overflow, however, is always available, and the quantity stored, as the water rises above the overflow, is certain to reduce to some extent the flood flows in the stream below. In the same way, storage in the ground is more or less available, depending on the condition of the ground-water at that time. The effect of storage above the overflow is to reduce all floods, and the effect of storage in the ground and that below the overflow is rather to make large floods of less frequent occurrence.

The temporary storage above the overflow is the most important for natural reservoirs, such as lakes and ponds. Where reservoirs are controlled and used for power purposes, or for water supply, the storage below the overflow may be important; but as these reservoirs may be filled when the larger floods come, dependence cannot be placed on such storage.

It is evident that this storage will affect the floods resulting from different types of storms in different ways. A storm producing a

long-continued flood will fill the available storage while the supply is still at a high rate, so that the maximum rate for the flood may be nearly as great as would have occurred without the existence of the reservoir. On the other hand, a flood from a short, sharp storm will be greatly reduced by storage, because, before the storage is filled, the supply will be much less.

In order to ascertain what effect this storage may have on the maximum rate of flood, a few specific cases were considered.

TYPES OF FLOOD FLOWS CONSIDERED

FOR SHOWING EFFECT OF STORAGE ON A STREAM

OF 1 SQ. MILE CATCHMENT AREA

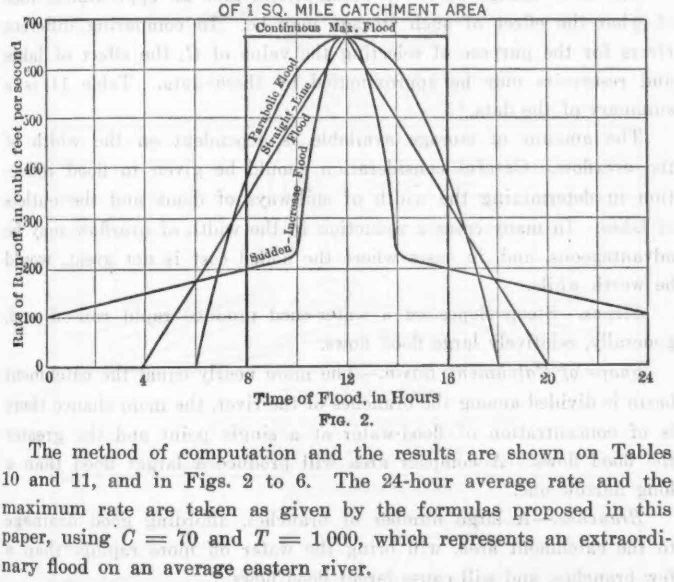


FIG. 2.

The method of computation and the results are shown on Tables 10 and 11, and in Figs. 2 to 6. The 24-hour average rate and the maximum rate are taken as given by the formulas proposed in this paper, using $C = 70$ and $T = 1\,000$, which represents an extraordinary flood on an average eastern river.

Four different types of flood flows are considered, as shown on Fig. 2. These cover in a general way the more common floods. The ideal type, called the Continuous Maximum, gives the minimum limit for the effect of storage. In these computations it is assumed that the reservoir sides are vertical and that the overflow is from a weir with vertical sides, from which the rate of overflow varies as the $\frac{3}{2}$ power of the height of the water passing over the weir.

Figs. 3, 4, and 5 show the effect of different amounts of storage for the assumed conditions. Table 10 contains a summary of the results of the computations for different cases.

Fig. 6 shows a plotting of the data in Table 10.

From these tables and diagrams the effect of this storage for the assumed conditions is apparent. The study is made for the effect on comparatively small catchment areas for which the flood at a high rate would usually not last more than 24 hours. For larger rivers, for which long-continued storms are common, the study does not apply.

The data, though based on assumptions, give an approximate idea of what the effect of such storage may be. In comparing different rivers for the purpose of selecting the value of C, the effect of lakes and reservoirs may be approximated by these data. Table 11 is a summary of the data.

The amount of storage available is dependent on the width of the overflow. Careful consideration should be given to flood reduction in determining the width of spillways of dams and the outlets of lakes. In many cases a reduction in the width of overflow may be advantageous, and, in cases where the added cost is not great, would be worth while.

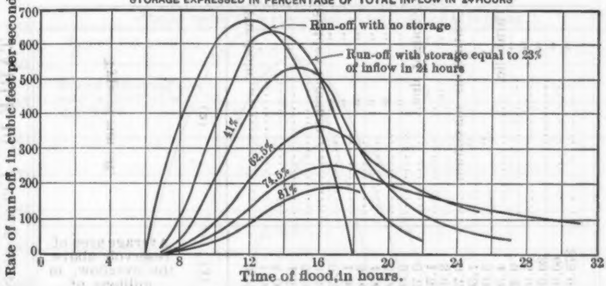
Slopes.—Steep slopes on a water-shed produce rapid run-off and, generally, relatively large flood flows.

Shape of Catchment Basin.—The more nearly equal the catchment basin is divided among the branches of the river, the more chance there is of concentration of flood-water at a single point and the greater the flood flows. A compact area will produce a larger flood than a long narrow one.

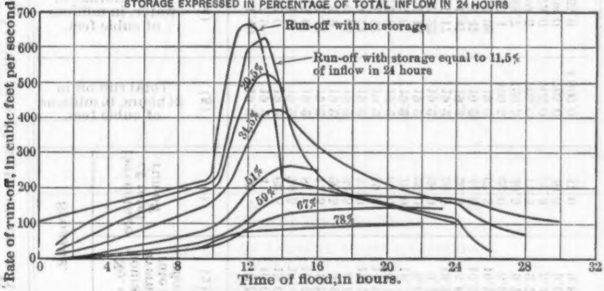
Branches.—A large number of branches, affording good drainage to the catchment area, will bring the water off more rapidly than a few branches, and will cause larger flood flows.

Condition of River-Bed.—The condition of the river-bed itself is an important factor. A river-bed which has frequent points of congestion which tend to hold back the water and provide temporary storage will have smaller relative flood flows than one with a smooth even bed which allows the water to run off rapidly. The hydraulic properties of the river channel, such as the hydraulic radius and the coefficient of roughness, affect the rapidity of the run-off, and thus

EFFECT OF STORAGE ABOVE OVERFLOW
PARABOLIC RUN-OFF
CATCHMENT AREA 1 SQ.MILE
STORAGE EXPRESSED IN PERCENTAGE OF TOTAL INFLOW IN 24 HOURS



EFFECT OF STORAGE ABOVE OVERFLOW
SUDDEN-INCREASE RUN-OFF
CATCHMENT AREA 1 SQ.MILE
STORAGE EXPRESSED IN PERCENTAGE OF TOTAL INFLOW IN 24 HOURS



EFFECT OF STORAGE ABOVE OVERFLOW
STRAIGHT-LINE RUN-OFF
CATCHMENT AREA 1 SQ.MILE
STORAGE EXPRESSED IN PERCENTAGE OF TOTAL INFLOW IN 24 HOURS

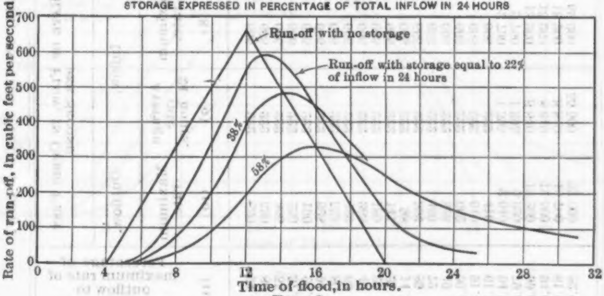


FIG. 3.

TABLE 10.—EFFECT OF STORAGE ABOVE THE OVERFLOW OF A RESERVOIR, ON THE FLOOD FLOW.

Catchment area, in square miles.	Type of run-off.	Average area of reservoir above the overflow, in millions of square feet.	Total storage of water, in millions of cubic feet.	Total run-off in 24 hours, in millions of cubic feet.	Storage.		Rate of Flow, in Cubic Feet per Second.			Percentage of maximum rate of outflow to maximum rate of inflow.
					Percentage of total run-off.	No. of hours at average rate of inflow.	Inflow.		Outflow.	
							Maximum rate.	Average rate 24 hours.	Maximum rate.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	Parabolic	0.75	4.40	19.2	23	5.5	666	222	630	94.5
1	"	1.50	7.9	19.2	41	10.0	666	222	530	80
1	"	3.00	12.0	19.2	62.5	15.0	666	222	365	55
1	"	4.50	14.3	19.2	74.5	18.0	666	222	250	37.5
1	"	6.00	15.6	19.2	81	19.5	666	222	190	28.5
1	Sudden increase	0.38	2.17	19.2	11.5	2.8	666	222	625	94
1	" "	0.75	3.9	19.2	20.5	4.9	666	222	535	80
1	" "	1.50	6.65	19.2	34.5	8.3	666	222	422	63
1	" "	3.00	9.74	19.2	51.0	12.2	666	222	265	40
1	" "	4.50	11.37	19.2	59.0	14.2	666	222	180	27
1	" "	6.00	12.9	19.2	67.0	16.1	666	222	141	21
1	" "	9.00	15.0	19.2	78.0	18.7	666	222	98	14.5
1	Straight line	0.75	4.14	19.2	22.0	5.3	666	222	592	89
1	" "	1.5	7.22	19.2	38.0	9.1	666	222	480	72
1	" "	3.0	11.17	19.2	58.0	13.9	666	222	326	49
1	Continuous maximum	0.75	4.47	19.2	23.5	5.6	666	222	662	99.5
1	" "	1.50	8.47	19.2	44.0	10.5	666	222	610	91.5
1	" "	3.00	12.98	19.2	67.5	16.2	666	222	410	61.5
10	Parabolic	4.56	26.4	120.5	22.0	5.3	2 790	1 395	2 640	94.0
10	"	9.45	48.7	120.5	40.5	9.7	2 790	1 395	2 220	79.5
100	"	15.0	89.5	765.0	11.5	2.76	13 260	8 840	13 200	99.0
100	"	30.0	173.0	765.0	22.5	5.4	13 260	8 840	12 500	94.0
100	"	60.0	308.0	765.0	40.0	9.6	13 260	8 840	10 700	81.0
1 000	"	375.0	1 965.0	4 800.0	41.0	9.8	69 300	55 500	57 000	82.0

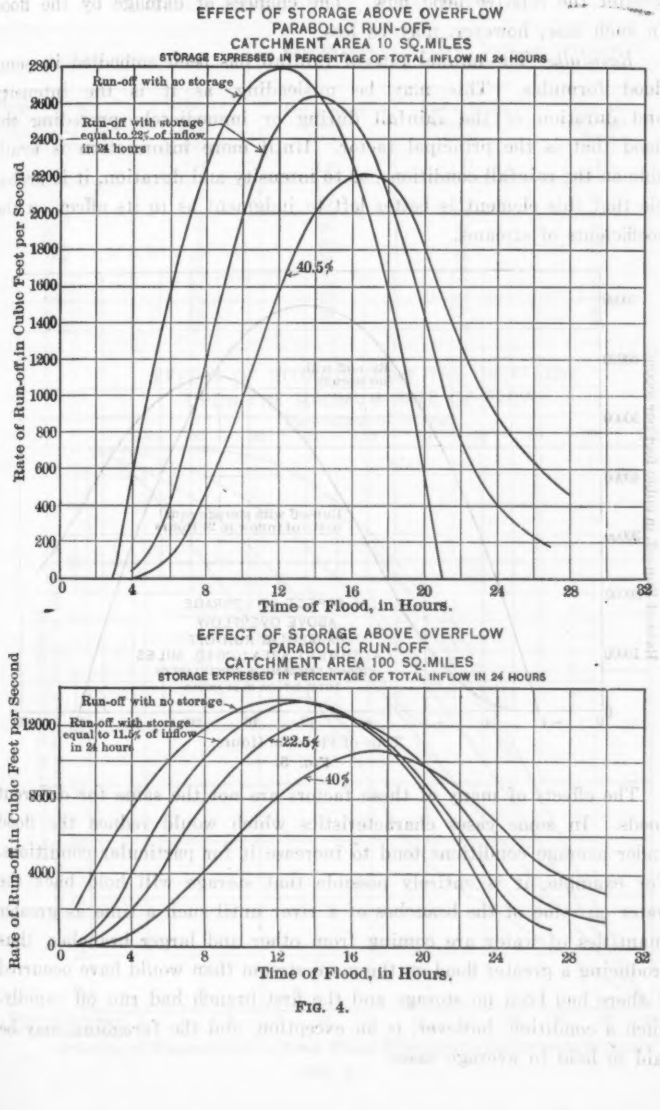

EFFECT OF STORAGE ABOVE OVERFLOW
PARABOLIC RUN-OFF
CATCHMENT AREA 10 SQ.MILES
STORAGE EXPRESSED IN PERCENTAGE OF TOTAL INFLOW IN 24 HOURS
Run-off with no storage
Run-off with storage equal to 22% of inflow in 24 hours
40.5%
Rate of Run-off, in Cubic Feet per Second
Time of Flood, in Hours.
EFFECT OF STORAGE ABOVE OVERFLOW
PARABOLIC RUN-OFF
CATCHMENT AREA 100 SQ.MILES
STORAGE EXPRESSED IN PERCENTAGE OF TOTAL INFLOW IN 24 HOURS
Run-off with no storage
Run-off with storage equal to 11.5% of inflow in 24 hours
22.5%
40%
Rate of Run-off, in Cubic Feet per Second
Time of Flood, in Hours.

FIG. 4.

the magnitude of the flood. The better the hydraulic properties, the greater the relative flood flow. The chances of damage by the flood in such case, however, may be less.

Rainfall.—The annual rate of rainfall has been embodied in some flood formulas. This may be misleading, as it is the intensity and duration of the rainfall during or immediately preceding the flood that is the principal factor. Until more information is available on the rainfall conditions, as to intensity and duration, it is probable that this element is better left to judgment as to its effect on the coefficients of streams.

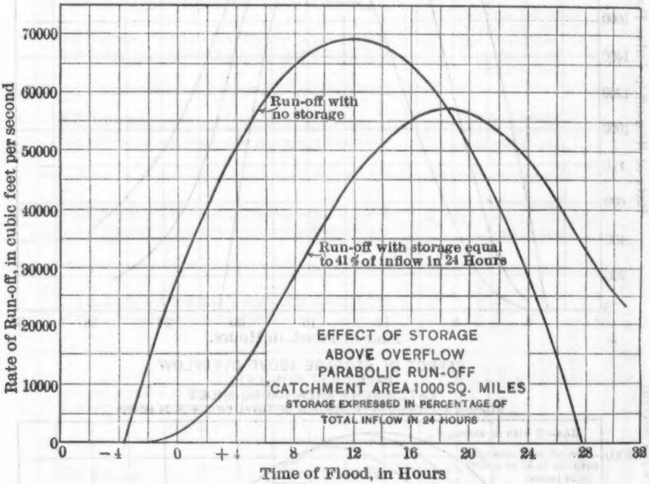


FIG. 5.

The effects of many of these factors are not the same for different floods. In some cases characteristics which would reduce the flood under average conditions tend to increase it for particular conditions. For example, it is entirely possible that storage will hold back the water on some of the branches of a river until such a time as greater quantities of water are coming from other and larger branches, thus producing a greater flood on the main stream than would have occurred if there had been no storage and the first branch had run off rapidly. Such a condition, however, is an exception, and the foregoing may be said to hold in average cases.

TABLE 11.—Effect of Storage Above the Overflow on the Maximum Rate of Flow, Based on Fig. 6.

Amount of Storage.		Percentage of Maximum Rate of Outflow to Maximum Rate of Inflow.		
In percentage of total water flowing into reservoir in 24 hours. (1)	In hours at average rate of inflow. (2)	For a short, sharp flood. (3)	For a long-continued flood. (4)	For an average flood. (5)
8 1/3	2	95	99	97
16 2/3	4	85	98	94
25	6	75	97	90
33 1/3	8	64	96	85
41 2/3	10	59	93	75
50	12	41	85	65
58 1/3	14	29	77	55
66 2/3	16	22	64	45
75	18	16	56	35
83 1/3	20	11	34	25

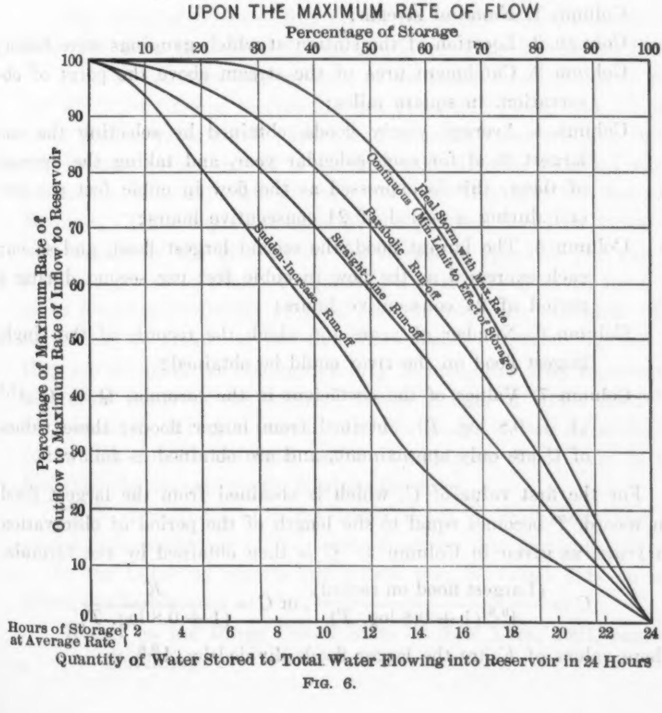


Fig. 6.

Values for the Coefficient, C, in the Formula, $Q = C\,A^{0.8}\ (1 + 0.8 \log. T)$.

Tables 12 to 26 contain flood data on the rivers in the United States for which the records could be obtained with sufficient completeness to give an approximation of the coefficient, C, in each case. Rivers for which observations are being taken at present, with records covering 4 years or more, are included, although, for these short-time periods, the value of the coefficient may be considerably in error, and should be adjusted when more complete records become available.

Each table contains a group of rivers corresponding to the classification made by the U. S. Geological Survey, and the arrangement of the rivers is similar to that given in the more recent papers of the Survey, in order that reference can readily be made thereto.

The tables contain the following data:

Column 1. Name of stream;

Column 2. Location of the station at which gaugings were taken;

Column 3. Catchment area of the stream above the point of observation, in square miles;

Column 4. Average yearly floods, obtained by selecting the one largest flood for each calendar year, and taking the average of these; this is expressed as the flow in cubic feet per second during a period of 24 consecutive hours;

Column 5. The largest flood, the second largest flood, and so on; each expressed as the flow in cubic feet per second during a period of 24 consecutive hours;

Column 6. Number of years for which the records of the single largest flood on the river could be obtained;

Column 7. Values of the coefficient in the formula, $Q = C\,A^{0.8}$ $(1 + 0.8 \log. T)$, obtained from larger floods; these values of C are only approximate, and are obtained as follows:

For the first value of C, which is obtained from the largest flood on record, T becomes equal to the length of the period of observation in years, as given in Column 6. C is then obtained by the formula,

$$C = \frac{\text{(Largest flood on record)}}{A^{0.8}\,(1 + 0.8 \log. T)},\ \text{or}\ C = \frac{K}{(1 + 0.8 \log. T)},$$

where values of K are the larger floods divided by $A^{0.8}$.

For the second value of C, Q becomes the average of the largest and the second largest floods; and the value of T becomes one-half of the period of observation, or

$$C = \frac{K + K_2}{1 + 0.8 \log. \left(\frac{1}{2}T\right)}.$$

For the third value of C, Q becomes the average of the three largest floods, and T becomes one-third the period of observation, or

$$C = \frac{K + K_2 + K_3}{1 + 0.8 \log. \left(\frac{1}{3}T\right)}.$$

Other values of C are obtained in a similar manner.

Column 8. Value of coefficient, C, as obtained from the average yearly flood; $C = \frac{Q\ (\text{Ave.})}{A^{0.8}}$.

This value of C, depending on all the yearly floods, is more accurate than those in Column 7, and should generally be selected for use.

Table 27 contains data on some of the larger single floods on rivers for which continuous records are not at hand. These data are taken from various reports and papers, and in many cases are only roughly approximate estimates. It is assumed that the flow recorded was the maximum rate, although in some instances this may not have been the case. These represent some of the greatest floods on record, but not necessarily the greatest on the stream in question for any considerable period of time. Frequently, a stream is observed for a year or so for some particular purpose, and the floods are recorded. These floods may be greater or less than the normal, but, from the fact that they are recorded, the assumption may be made that they represent the maximum floods on the stream. Tables of data of single floods may be misleading for this reason. In this study, it was in some cases found that flows taken from tables of maximum floods were actually less than the average yearly flood on the river.

Table 27 includes only the larger floods of this kind; and they probably represent one of the larger floods on that particular river.

In the following reports and papers may be found data on many rivers not here given:

Report on the Barge Canal, State of New York, 1901, page 844, Kuichling.

Hydrology of the State of New York, Rafter.

Geological Survey of New Jersey, 1904, Vermeule.

Reports of Pennsylvania Water Supply Commission.

Reports of New York State Water Supply Commission.

Reports of Maine State Water Storage Commission.

United States Geological Survey, Water Supply Paper No. 147, page 184.

United States Geological Survey, Water Supply Paper No. 162, "Destructive Floods."

Selecting the Value of T, in Design.—Floods have occurred on some rivers during the last 20 years which, normally, would be repeated in not less than 1 000 years. If works are to provide for floods equal to the greatest that have been observed, a value of T of at least 1 000 should be used. Such a flood or a greater one may occur on any river at any time, but it is not likely to come soon on any particular stream. It must be remembered that the use of $T = 1\,000$ does not mean that the corresponding flood will come at the end of 1 000 years, but that the chances are even that it will occur at some time during a period of 1 000 years. It means, also, that the chances are 1 to 1 000 that it will occur in any one year, or 1 to 100 that it will occur in 10 years, or 1 to 10 that it will occur in a century.

The selection of the proper value of T then becomes a question of what chance we can afford to take. In the design of a spillway of an earth dam, the failure of which would cause great damage, a large value of T should be used. The design of a temporary dam for construction purposes, the failure of which would mean only small damage, would call for the use of a comparatively small value of T.

The decision as to the value of T to be selected should depend on considerations of the first cost of the construction required to provide for it, together with the probable damage in case the flood exceeds the quantity provided for. The added cost due to using a larger value of T may be regarded as insurance against loss. Obviously, if insurance is costly, as compared to the risk, less should be taken. Consideration of possible loss of life and property, together with loss of business, or loss of a water supply for a city, and other factors, must be taken into account in deciding on the proper value of T. Possible increased damage in the future, due to building up of sections below the dam, or unusual difficulties in reconstruction, must be considered.

TABLE 12.—Values for Coefficient, C, in Formula, $Q = C\ A^{0.8}\ (1 + 0.8 \log. T)$. New England States.

Name of stream. (1)	Location of station. (2)	Catchment area, in square miles, A. (3)	Flood Flows, in Cubic Feet per Second. 24-Hour Average. Average yearly, Q (Ave.). (4)	Flood Flows, in Cubic Feet per Second. 24-Hour Average. Larger flood, Q. (5)	Period of observation. T. (6)	Values of C. From larger floods. (7)	Values of C. From average yearly flood. (8)
St. John	Fort Kent, Me.	5 280	62 500	75 600	6	49	66
Fish	Wallagrass, Me.	890	7 339	8 970	5	14	17
Aroostook	Fort Fairfield, Me.	2 230	30 200	34 300	6	44	63
St. Croix	Woodland, Me.	1 420	9 600	20 300	9	35	29
Machias	Whitneyville, Me.	465	5 740	11 100	8	47	42
Penobscot (West Branch)	Millinockett, Me.	1 880	14 000	24 250	11	32	34
				22 110		36	
Penobscot	West Enfield, Me.	6 600	60 630	93 400	11	44	53
				91 400		50	
Penobscot (East Branch)	Grindstone, Me.	1 100	12 600	21 400	9	47	47
Mattawamkeag	Mattawamkeag, Me.	1 500	16 000	24 400	9	40	46
				21 100		43	
Piscataquis	Foxcroft, Me.	286	10 170	18 100	9	112	110
				13 800		114	
Kennebec	The Forks, Me.	1 570	13 720	19 890	11	30	38
				18 330		33	
Kennebec	Bingham, Me.	2 660	32 000	48 000	8	51	58
				37 750		53	
Kennebec	Waterville, Me.	4 270	59 600	151 000	18	94	74
				111 200		93	
				86 200		89	
Roach	Roach, Me.	85	1 660	1 970	7	33	47
Carrabasset	North Anson, Me.	340	9 080	13 670	5	83	86
Sandy	Madison, Me.	650	9 800	13 800	5	50	55
Cobbossecontec	Gardiner, Me.	240	1 850	3 275	21	20	23
				3 200		22	
				2 700		23	
Androscoggin	Rumford Falls, Me.	2 090	24 900	55 500	40	53	55
				55 200		59	
				39 000		57	
Saco	Center Conway, N. H.	385	10 500	14 100	6	75	90
Pemigewasset	Plymouth, N. H.	615	16 800	30 640	24	86	99
				23 400		86	
				22 760		88	
Merrimac	Franklin Junction, N. H.	1 460	18 700	27 900	7	50	55
Merrimac	Garvins Falls, N. H.	2 340	24 600	32 900	7	39	45
Merrimac	Lawrence, Mass.	4 638	43 400	82 150	56	40	50
				74 000		42	
				65 000		42	
				62 500		42	
				61 200		44	
Connecticut	Orford, Vt.	3 305	31 700	49 700	11	42	49
				40 600		44	
Connecticut	Sunderland, Mass.	7 700	78 700	103 000	8	47	61
				92 200		52	
Connecticut	Holyoke, Mass.	8 144	73 000	115 000	26	40	54
				112 000		45	
				99 700		46	
				94 300		47	
				87 900		48	
Connecticut	Hartford, Conn.	10 234	113 400	205 200	104	49	71
				192 800		52	
				178 400		54	
				175 000		55	
				169 300		56	
				167 000		57	

TABLE 12.—(*Continued.*)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Israel (above South Branch)	Jefferson Highland, N. H.	8.7	356	554	3	71	63
Israel (below South Branch)	Jefferson Highland, N. H.	21.2	721	1 050	4	61	63
Fomer	Above Reservoir, Holyoke, Mass.	13	434	788	14	53	56
				672		56	
				628		58	
Westfield Little	Blandford, Mass.	43	1 420	2 135	4	71	70
Housatonic	Gaylordsville, Conn.	1 020	16 000	31 000	9	69	63
				25 700		73	

TABLE 13.—VALUES FOR COEFFICIENT, C, IN FORMULA, $Q = C\ A^{0.8}\ (1 + 0.8 \log. T)$.

HUDSON RIVER BASIN.

Name of stream. (1)	Location of station. (2)	Catchment area, in square miles, A. (3)	Flood Flows, in Cubic Feet per Second, 24-Hour Average. Average yearly. Q (Ave.). (4)	Larger flood. Q. (5)	Period of observation. T. (6)	Values of C. From larger floods. (7)	From average yearly flood. (8)
Hudson	Fort Edward, N. Y.	2 800	32 900	43 900	13	40	58
				42 800		46	
				42 600		53	
Hudson	Mechanicsville, N. Y.	4 500	41 500	70 000	40	36	49
				59 400		38	
				56 300		39	
				54 900		41	
Hoosic	Buskirk, N. Y.	579	10 024	13 680	5	54	62
Mohawk	Little Falls, N. Y.	1 310	20 330	24 220	7	46	65
				23 500		53	
Mohawk	Dunsbach Ferry, N. Y.	3 440	50 500	84 200	12	68	75
				59 200		66	
				55 700		67	
West Canada Creek	TwinRock Bridge, N. Y.	364	13 050	34 350	9	171	116
				16 150		147	
East Canada Creek	Dolgeville, N. Y.	256	5 950	12 150	12	78	71
				7 540		72	
Schoharie	Prattsville, N. Y.	240	7 940	13 100	9	93	99
				12 860		107	
Esopus	Olive Bridge, N. Y.	239	8 420	15 388	7	114	105
				9 376		108	
Esopus	Kingston, N. Y.	324	13 160	20 500	9	114	129
				18 700		126	
Rondout	Rosendale, N. Y.	380	13 290	19 510	9	96	115
				18 300		107	
Reels Cr. and Johnson Br.	Durfield, N. Y.	4.42	243	296	4	61	74
Starch Factory	Hartford, N. Y.	3.4	351	515	4	131	132
Graefenburg Creek	Hartford, N. Y.	0.282	13.4	15.2	4	28	37

Although the selection of the proper value of T must be a matter of engineering judgment, the following method will give the economic value: Estimate the probable amount necessary to make good all damage caused by the flood exceeding the allowed amount. Multiply this by the probability of the flood occurring in one year. The product

TABLE 14.—Values for Coefficient, C, in Formula, $Q = C\,A^{0.8}\ (1 + 0.8 \log. T)$.

Middle Atlantic States.

Name of stream.	Location of station.	Catchment area, in square miles, A.	Flood Flows, in Cubic Feet per Second. 24-Hour Average. Average yearly, Q (Ave.).	Larger flood. Q.	Period of observation, T.	Values of C. From larger floods.	From average yearly flood.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Passaic	Chatham, N. J.	101	1 914	2 986 2 860	9	43 48	48
Passaic	Dundee Dam, N. J.	823	10 600	27 995 21 370 18 200 16 600	34	58 58 57 57	50
Raritan	Bound Brook, N. J.	800	19 700	28 500	6	83	93
Delaware (East Branch)	Hancock, N. Y.	920	32 900	72 500 50 500	9	175 172	140
Delaware	Port Jervis, N. Y.	3 250	62 500	108 000 67 300	8	98	97
Delaware	Riegelsville, N. J. (Former Station at Lambertville, N. J.)	6 430	99 000	176 900 171 700 158 300	15	82 92 96	89
Delaware (West Branch)	Hancock, N. Y.	680	19 250	33 740 31 830	9	105 117	105
Musconetcong	Bloomsbury, N. J.	146	1 810	2 780	4	34	34
Tohickon Creek	Point Pleasant, Pa.	102	4 120	8 650 6 515 5 960 5 360	25	102 101 101 101	103
Neshaminy Creek	Low Forks, Pa.	139	4 620	9 012 8 707 6 985 6 063	27	81 90 90 88	89
Perkiomen	Frederick, Pa.	152	5 020	8 769 7 051 6 843 6 789	27	73 74 77 79	89
Schuylkill	Near Philadelphia, Pa.	1 920	30 400	82 156 36 600 36 180	14	101 84 80	72
Susquehanna	Binghamton, N. Y.	2 400	39 100	63 000 48 900	10	70 71	78
Susquehanna	Wilkes-Barre, Pa.	9 810	123 800	217 700 166 300 153 000	12	75 76 78	79
Susquehanna	Danville, Pa.	11 100	143 250	304 800 228 400 176 000	12	95 93 91	83
Susquehanna	Harrisburg, Pa.	24 000	276 000	593 000 543 000 484 000 405 000	21	90 98 100 100	87
Susquehanna	McCall's Ferry, Pa.	26 800	286 000	352 900	7	60	82
Chenango	Binghamton, N. Y.	1 530	25 970	35 900 34 600 31 200	11	56 63 67	74
Chemung	Chemung, N. Y.	2 440	35 740	46 200 42 900	7	53 60	69

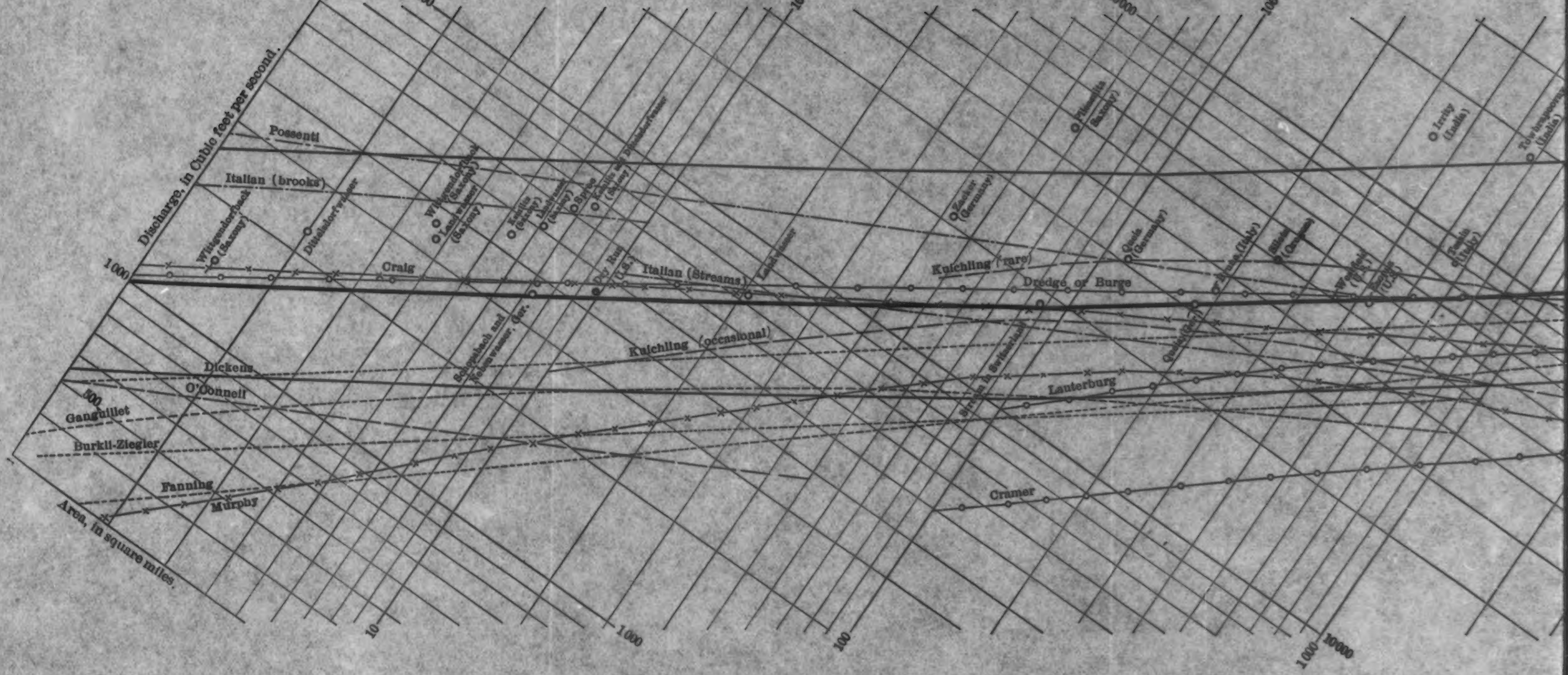
TABLE 14.—(*Continued.*)

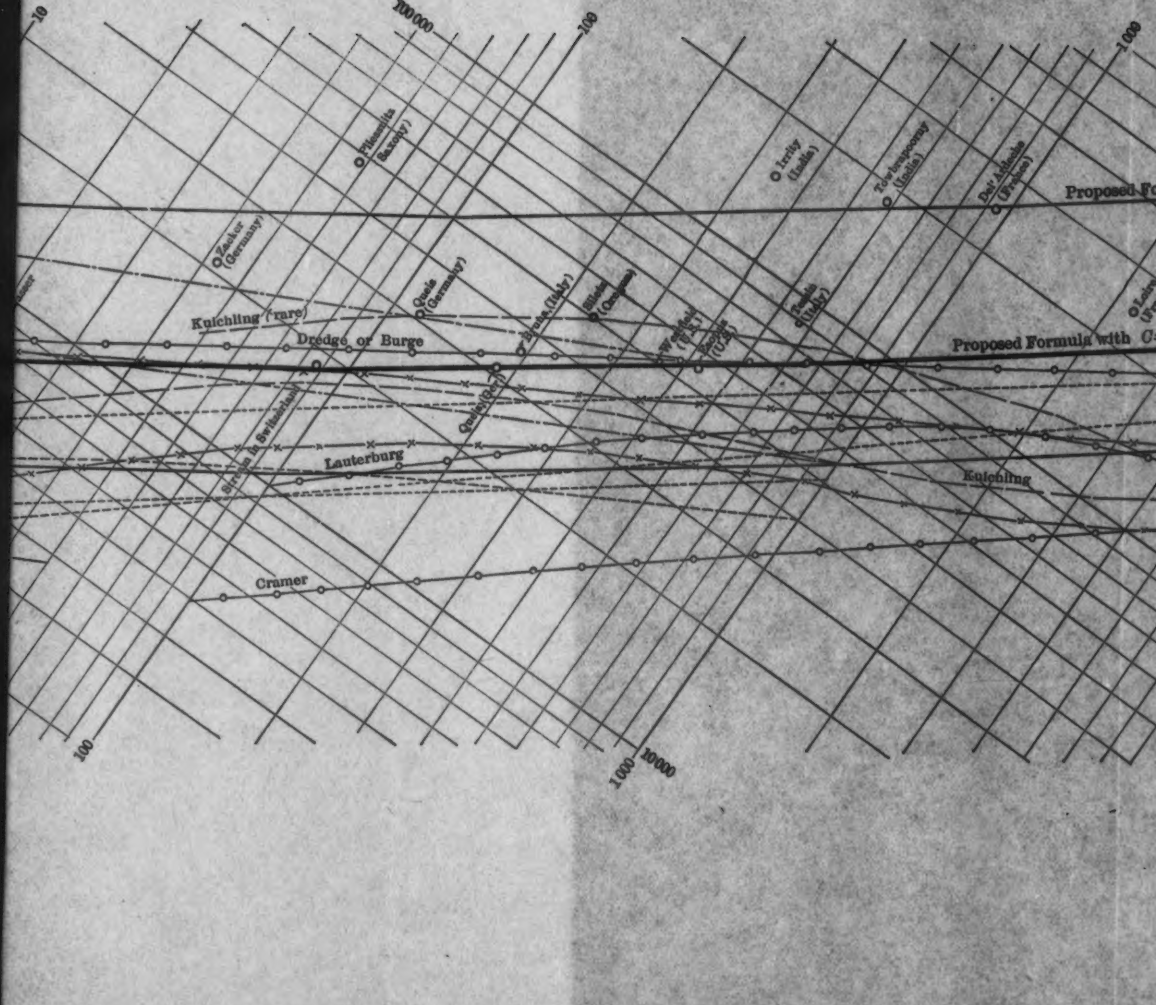
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Susquehanna (West Branch)	Williamsport, Pa	5 640	104 300	164 100	17	82	100
				162 600		94	
				150 900		99	
Juniata	Newport, Pa	3 480	63 500	118 000	11	95	93
				102 000		103	
Broad Creek	Millgreen, Md	16.4	279	509	5	35	30
Gunpowder Falls	Glencoe, Md	160	3 780	5 530	4	65	65
Little Gunpowder Falls	Bel Air, Md	43	898	1 600	4	53	44
Patapsco	Woodstock, Md	251	6 890	11 000	12	72	83
				9 700		77	
Potomac	Point of Rocks, Md	9 650	114 000	218 700	17	72	67
				203 800		79	
				182 000		82	
Shenandoah	Millville, W. Va	3 000	44 800	139 700	13	122	74
				77 900		109	
				60 800		101	
Potomac (North Branch)	Piedmont, W. Va	410	8 110	13 450	7	65	67
Antietam Creek	Sharpsburg, Md	295	3 240	6 835	8	42	34
				4 110		39	
Shenandoah (South Fork)	Front Royal, Va	1 570	25 200	76 800	7	125	70
Monocacy	Frederick, Md	660	14 800	20 460	15	59	82
				19 200		65	
				17 400		68	

is the annual value of the flood risk. Estimate the first cost of the works necessary to provide for floods of a magnitude corresponding to these values for T. Multiply this by the going rate of interest. The product is the annual interest charge. Select the value of T for which the sum of the annual value of the flood risk and the annual interest charge is a minimum.

Comparison with Other Flood Formulas.

Table 29 is a list of some other formulas for obtaining the maximum flood flow. These formulas are of different forms, and contain many different terms. Some are intended only for use over a limited section, and others are for more general use. In order to make any comparison, it is necessary to assume values for the variable terms. The values assumed are intended to suit average conditions in the eastern portion of the United States. Three of the formulas were derived for such conditions, and the others were for European and Indian conditions. For the purpose of comparison, data on some of the larger floods on foreign streams are given in Table 30. On Plate XIII the formulas in Table 29 are plotted, together with three curves representing the formula proposed in this paper. An examination of this diagram shows that formulas vary widely. This is to


Discharge, in Cubic feet per second.
Area, in square miles.
Possenti
Italian (brooks)
Craig
Italian (Streams)
Kuichling (rare)
Dredge or Burge
Kuichling (occasional)
Dickens
O'Connell
Ganguillet
Burkli-Ziegler
Fanning
Murphy
Lauterburg
Cramer
1000
500
10000
1000
10
100
10000



10
100000
100
1000
Pliessnitz (Saxony)
Irrity (India)
Tovbrapoorny (India)
Del'Ardeche (France)
Proposed Fo
Zacker (Germany)
Queis (Germany)
Kuichling (rare)
Dredge or Burge
Bruna (Italy)
Proposed Formula with C
Queis (Ger.)
Streams in Switzerland
Lauterburg
Kuichling
Cramer
100
1000
10000

PLATE XIII.
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FULLER ON
FLOOD FLOWS.

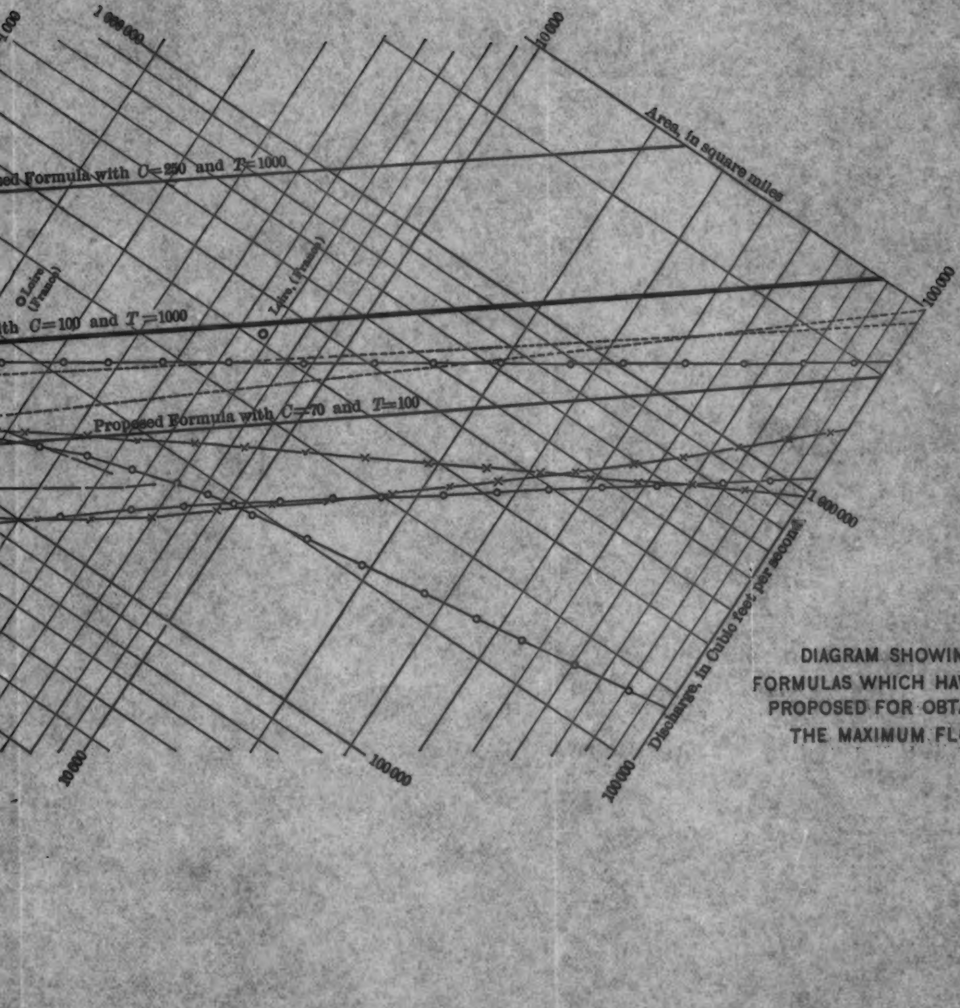


DIAGRAM SHOWING
FORMULAS WHICH HAVE BEEN
PROPOSED FOR OBTAINING
THE MAXIMUM FLOOD

TABLE 15.—VALUES FOR COEFFICIENT, C, IN FORMULA, $Q = C\,A^{0.8}\ (1 + 0.8 \log. T)$.

SOUTH ATLANTIC STATES.

Name of stream.	Location of station.	Catchment area in square miles, A.	Flood Flows, in Cubic Feet per Second. 24-Hour Average.		Period of observation. T.	Values of C.	
			Average yearly, Q (Ave.).	Larger flood, Q.		From larger floods.	From average yearly flood
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
James	Buchanan, Va	2 660	40 846	62 000	15	71	91
				52 565		78	
				51 400		82	
				48 620		86	
James	Cartersville, Va	6 230	61 658	84 800	12	42	57
				75 800		46	
James (North Fork)	Near Glasgow, Va	831	16 600	37 250	10	95	77
				24 060		100	
Appomattox	Mattoax, Va	745	8 449	11 695	4	40	43
Roanoke	Roanoke, Va	388	8 813	18 104	14	80	75
				12 472		84	
Dan	South Boston, Va	2 750	23 140	44 400	5	50	41
Cape Fear	Fayetteville, N. C.	4 493	52 800	90 650	15	56	63
				71 600		60	
				69 700		64	
Yadkin	North Wilkesboro, N. C.	500	12 345	17 900	7	74	85
				15 700		82	
Yadkin	Salisbury, N. C.	3 400	62 192	130 000	15	100	93
				115 000		111	
				79 998		116	
Catawba	Morganton, N. C	758	20 839	32 200	8	93	103
Catawba	Catawba, N. C	1 535	29 215	61 050	6	106	82
Catawba	Rockhill, S. C	2 987	60 000	150 783	6	154	99
Wateree	Camden, S. C	4 500	30 690	36 500	6	27	37
Broad (of the Carolines)	Alston, S. C	4 610	76 400	131 000	11	84	89
				130 500		96	
Saluda (Branch Mobile)	Waterloo, S. C	1 056	18 800	18 850	9	41	53
				18 500		47	
Tugalloo (Branch Savannah)	Madison, S. C	593	15 301	21 860	10	73	93
				17 300		80	
Savannah	Augusta, Ga	7 300	114 300	309 980	20	124	93
				253 000		133	
				220 000		130	
Tallulah	Tallulah Falls, Ga	191	4 870	9 000	7	80	73
Broad (of Georgia)	Carlton, Ga	762	20 428	47 200	18	123	101
				29 125		114	
Ocmulgee	Jackson, Ga	1 400	17 920	25 400	5	50	54
Ocmulgee	Macon, Ga	2 420	32 550	50 860	13	50	64
				46 240		55	
				43 100		57	
Oconee	Greensboro, Ga	1 100	18 540	68 200	8	145	68
Oconee	Milledgeville, Ga	2 840	16 600	30 000	5	35	29
Oconee	Dublin, Ga	4 180	29 013	37 000	13	25	37
				36 600		28	
Appalachee	Buckhead, Ga	440	5 020	7 660	8	34	39
Chattahoochee	Norcross, Ga	1 170	17 747	30 180	8	62	62
Chattahoochee	Oakdale, Ga	1 560	34 100	48 800	8	79	95
Chattahoochee	West Point, Ga	3 300	48 483	88 630	14	70	74
				66 090		75	
Soquee	Demorest, Ga	158	5 652	8 830	5	93	93
Flint	Woodbury, Ga	990	16 434	30 250	10	67	66
				25 750		75	
Flint	Montezuma, Ga	2 700	14 320	20 800	5	24	25
Flint	Albany, Ga	5 000	29 731	42 600	9	27	33
				38 970		30	

TABLE 15.—(*Continued.*)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Pea	Pera, Ala.	1 180	9 643	12 600	6	27	33
Conecuh	Beck, Ala.	1 290	10 600	16 400	6	33	35
Coosawattee (Branch Mobile)	Carters, Ga.	531	12 500	17 700	12	63	83
				16 950		72	
Oostanaula (Branch Mobile).	Resaca, Ga.	1 610	23 661	39 200	11	59	64
				36 600		65	
Coosa	Rome, Ga.	4 006	54 100	64 186	7	48	71
Coosa (Branch Mobile)	Riverside, Ala.	7 060	57 562	75 800	14	33	48
				75 000		38	
				72 160		41	
Alabama	Selma, Ala.	15 400	114 028	146 000	12	36	51
				145 700		40	
				138 000		44	
Etowah	Canton, Ga.	604	13 440	19 000	12	61	80
				17 090		68	
				16 094		72	
Etowah	Rome, Ga.	1 800	35 200	59 400	6	91	87
Choccolocco Creek	Jenifer, Ala.	272	5 110	11 800	4	90	58
Tallapoosa	Sturdevant, Ala.	2 500	36 247	59 100	10	63	69
				50 800		70	
				40 510		69	
Cahaba	Centerville, Ala.	1 040	11 981	17 100	6	41	46
				12 980		42	
Tombigbee	Columbus, Miss.	4 440	34 476	50 420	10	34	41
				46 800		39	
				40 498		39	
Tombigbee	Epes, Ala.	8 830	44 887	61 000	8	25	31
				50 500		28	
Black Warrior	Cordova, Ala.	1 900	42 100	56 900	9	77	100
				51 800		87	
Black Warrior	Tuscaloosa, Ala.	4 900	101 000	141 000	17	79	113
				137 000		89	
				129 000		97	
Pearl	Jackson, Miss.	3 120	21 649	36 500	8	34	35

be expected, as the data on which they are based are very different. On the whole, it may be said that the formula proposed in this paper gives relatively higher values of the flood flow for very small and very large catchment areas. Probably the reason for this is that the formula was derived from average floods, and many, if not all, of the others were derived from maximum floods. There are numerous records for medium-sized streams, but not for very small or very large ones. It is to be expected, under these conditions, that the highest relative maximum floods will be found on rivers of medium size. There are comparatively few very large rivers in Europe, and their number in America is limited. Although there are many small streams, damage from flood on them is not usually great, and records of their floods are comparatively few. More records of small streams are probably available on European than on American rivers.

Some of the largest floods on foreign and American streams are plotted on Plate XIII. The largest of these are on streams in India and Saxony. The upper curve, representing the proposed formula,

TABLE 16.—VALUES FOR COEFFICIENT, C, IN FORMULA, $Q = C\ A^{0.8}\ (1 + 0.8 \log. T)$.

OHIO RIVER BASIN.

Name of stream.	Location of station.	Catchment area, in square miles, A.	FLOOD FLOWS, IN CUBIC FEET PER SECOND. 24-HOUR AVERAGE. Average yearly, Q (Ave.).	Larger flood. Q.	Period of observation. T.	VALUES OF C. From larger floods.	From average yearly flood.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Allegheny	Red House, N. Y.	1 640	25 200	41 000	8	64	67
Allegheny	Kittanning, Pa.	8 690	126 000	232 000	7	97	88
Kiskiminetis	Avonmore, Pa.	1 750	46 400	77 700	4	133	118
Blacklick Creek	Blacklick, Pa.	403	9 668	15 400	7	75	79
Cheat	Morganton, W. Va.	1 380	19 540	30 400	8	54	60
				25 900		59	
Youghiogheny	Friendsville, Md.	294	6 660	8 160	6	54	71
Youghiogheny	Confluence, Pa.	435	11 580	24 000	8	108	90
Casselman	Confluence, Pa.	450	10 250	20 800	6	96	77
Laurel Hill Creek	Confluence, Pa.	118	2 851	5 020	5	74	67
Mahoning	Youngstown, Ohio	958	14 911	19 400	4	54	61
New	Radford, Va.	2 720	64 200	137 760	13	130	115
				123 800		142	
New	Fayette, W. Va.	6 200	78 155	110 527	7	61	72
				107 740		70	
Greenbrier	Alderson, W. Va.	1 340	36 900	62 450	14	108	117
				55 850		111	
				48 700		115	
Scioto	Columbus, Ohio	1 050	14 515	21 340	5	53	56
French Broad	Asheville, N. C.	987	16 400	30 720	11	67	65
				26 350		71	
Tennessee	Knoxville, Tenn.	8 990	92 891	157 410	10	60	64
				124 070		62	
Tennessee	Chattanooga, Tenn.	21 400	231 000	409 520	21	68	79
				381 040		74	
				363 240		78	
				363 240		82	
Mill (North Fork)	Pinkbed, N. C.	24	763	1 170	4	62	60
Mill (South Fork)	Sitton, N. C.	40.5	1 562	2 050	4	72	81
Pigeon	Newport, Tenn.	655	10 314	20 260	7	67	57
				12 300		63	
Nolichucky	Greeneville, Tenn.	1 100	14 093	32 800	6	74	52
				18 900		69	
Holston (South Fork)	Bluff City, Tenn.	828	20 130	32 980	6	93	92
				24 380		94	
Holston	Rogersville, Tenn.	3 060	37 400	51 400	5	53	60
Watauga	Elizabethton, Tenn.	408	6 013	9 530	4	52	49
Little Tennessee	Judson, N. C.	675	25 800	57 140	14	163	142
				50 460		180	
Little Tennessee	McGhee, Tenn.	2 470	37 240	70 000	6	73	63
Tuckaseegee	Bryson, N. C.	662	22 500	38 550	13	113	124
				36 400		125	
Hiwassee	Murphy, N. C.	410	12 300	22 360	13	96	101
				22 000		110	
Hiwassee	Reliance, Tenn.	1 180	28 550	55 200	10	109	101
				38 000		106	
Hiwassee	Charleston, Tenn.	2 297	37 055	45 940	4	63	76
Valley	Tomotla, N. C.	106	4 842	10 400	5	160	117
Nottenly	Ranger, N. C.	272	4 122	5 660	5	41	47
Toccoa	Blue Ridge, Ga.	231	7 958	12 290	5	102	102
Ocoee	McCays, Copperhill, Tenn.	374	7 000	18 000	6	97	61
Elk	Elkmont, Ala.	1 700	39 750	49 000	4	83	103

TABLE 16.—(*Continued.*)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Dock	Columbia, Tenn.	1 260	18 660	25 600	5	55	62
Clarion	Clarion, Pa.	910	23 280	39 300	20	82	98
				36 000		89	
				34 200		94	
				32 700		97	
Ohio	Wheeling, W. Va.	23 800	294 000	480 000	50	65	93
				460 000		70	
				430 000		67	
				428 000		67	
				428 000		69	

TABLE 17.—Values for Coefficient, C, in Formula, $Q = C\,A^{0.8}\ (1 + 0.8 \log.\ T)$.

St. Lawrence River Basin.

Name of stream. (1)	Location of station. (2)	Catchment area, in square miles, A. (3)	Flood Flows, in Cubic Feet per Second. 24-Hour Average. Average yearly Q (Ave.). (4)	Larger flood. Q. (5)	Period of observation. T. (6)	Values of C. From larger floods. (7)	From average yearly flood. (8)
Dead	Forestville, Mich.	142	2 270	2 340	4	30	43
Escanaba	Escanaba, Mich.	800	7 340	10 870	7	31	35
Menominee	Iron Mountain, Mich.	2 420	11 180	15 100	9	16.8	22
				15 000		19.4	
St. Joseph	Buchanan, Mich.	3 940	11 100	18 600	5	15.8	14.8
				11 470		15.2	
Kalamazoo	Allegan, Mich.	1 470	4 700	10 260	6	18.5	13.7
				5 219		16.4	
Muskegon	Newaygo, Mich.	2 350	4 740	6 266	4	8.4	9.5
Manistee	Sherman, Mich.	900	2 380	2 870	8	7.2	10.3
Thunder Bay	Alpena, Mich.	1 260	5 400	7 275	7	14.3	17.8
Au Sable	Bamfield, Mich.	1 420	3 470	4 220	6	7.9	10.4
Huron	Geddes, Mich.	757	2 290	3 734	7	11.1	11.4
Huron	Flat Rock, Mich.	1 000	2 310	2 780	7	6.6	9.2
Seneca	Baldwinsville, N. Y.	3 100	8 846	10 865	8	10.2	14
				10 310		11.5	
Oneida	Euclid, N. Y.	1 310	9 983	13 760	7	26	32
Chittenango Creek	Chittenango, N. Y.	79	1 390	1 922	5	37	42
Salmon	Pulaski, N. Y.	259	9 310	10 500	5	79	110
Black	Felts Mills, N. Y.	1 850	18 270	23 100	9	32	44
				19 200		35	
Moose	Moose River, N. Y.	346	5 780	6 760	11	35	54
				6 670		40	
Oswegatchie	Ogdensburg, N. Y.	1 580	11 990	15 800	8	26	33
Raquette	Massena Springs, N. Y.	1 170	8 540	11 000	8	21	30
				9 920		26	
Saranac	Plattsburg, N. Y.	624	4 120	4 680	6	16.6	24
Winooski	Richmond, Vt.	885	12 225	16 300	4	48	53
Genesee	Rochester, N. Y.	2 365	22 100	50 000	128	37	44
				40 000		37	
				36 500		38	
				36 000		37	
				36 000		37	
				33 000		37	
Genesee	Rochester, N. Y.	2 365	22 400	36 500	12	39	45
				36 000		45	
				28 300		45	
				28 000		47	

TABLE 18.—Values for Coefficient, C, in Formula, $Q = C\ A^{0.8}\ (1 + 0.8 \log. T)$.

Hudson Bay.

Name of stream.	Location of station.	Catchment area, in square miles, A	Flood Flows, in Cubic Feet per Second. 24-Hour Average. Average yearly, Q (Ave.).	Larger flood, Q.	Period of observation, T.	Values of C. From larger floods.	From average yearly flood.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
St. Mary (above Swiftcurrent Creek)	Near Babb, formerly Main, Mont	177	3 397	7 980	9	72	54
				4 360		64	
St. Mary	Near Cardston, Alta	452	5 945	18 000	8	79	45
				6 175		61	
Swiftcurrent Creek	Near Babb, Mont	101	2 248	4 540	9	65	57
				3 360		65	
Ottertail (head of Red)	Near Fergus Falls, Minn	1 310	813	1 075	7	2.0	2.6
Red	Fargo, N. Dak	6 020	3 086	6 089	9	3.2	2.9
				4 420		3.2	
Red	Grand Forks, N. Dak	25 000	21 706	32 920	8	5.8	6.6
				29 400		6.4	
Sheyenne	Haggart. N. Dak	5 400	1 300	2 030	5	1.3	1.3
Red Lake	Crookston, Minn	5 320	8 669	14 200	8	8.6	9.1
Pembina	Neche, N. Dak	2 940	1 938	3 870	7	3.9	3.3

with $C = 250$ and $T = 1\,000$, covers all except two of these floods. There is no record of any American flood as large as these, although the record of the Siletz River, covering a 3-year period, includes one of great magnitude; the maximum rate is not stated, but the 24-hour average rate is relatively greater than that of any flood recorded on any other American stream. It is stated by the U. S. Geological Survey that this stream is typical of those in that section of Oregon, and it seems probable that floods there equal or exceed those on foreign rivers. It is also probable that other floods for which no records are available have exceeded any of those recorded.

On the lower diagram of Plate XI are plotted the larger floods on the rivers in the eastern part of the United States. The two curves drawn to represent the formula, with $C = 100$, $T = 1\,000$, and $C = 100$, $T = 50$, cover these great floods well. These may be considered as extraordinary floods on streams which usually give large floods. These few floods represent the largest ones on many rivers during the last 50 or 100 years, so that some of them may be those

TABLE 19.—VALUES FOR COEFFICIENT, C, IN FORMULA, $Q = C A^{0.8} (1 + 0.8 \log. T)$.

UPPER MISSISSIPPI RIVER BASIN.

Name of stream.	Location of station.	Catchment area, in square miles, A.	Flood Flows, in Cubic Feet per Second. 24 Hour Average. Average yearly. Q (Ave.).	Larger flood. Q.	Period of observation, T.	Values of C. From larger floods.	From average yearly flood.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Mississippi	Above Sandy River, Minn.	4 510	6 250	9 572	15	5.9	7.4
				8 823		6.5	
				8 299		6.8	
Mississippi	Sauk Rapids, Minn........	12 400	32 400	51 000	3	19.6	17
Mississippi	Anoka, Minn............	17 100	31 450	44 300	6	11.2	13
				36 600		12.0	
Mississippi	St. Paul, Minn..........	35 700	42 223	80 800	19	9.2	9.7
				73 000		9.9	
				59 800		10.0	
				58 800		10.0	
Pine	Below Pine River Reservoir, Minn.	452	1 051	1 586	16	6.1	7.9
				1 520		6.8	
				1 479		7.3	
Minnesota.........	Mankato, Minn...........	14 600	24 035	43 800	8	11.6	11.0
Chippewa	Chippewa Falls, Wis......	5 300	36 454	64 400	11	37	38
				45 900		36	
Chippewa	Eau Claire, Wis..........	6 740	43 105	60 520	6	32	37
Flambeau	Ladysmith, Wis..........	2 120	10 079	12 750	4	18.9	21
Black.............	Neillsville, Wis..........	675	10 000	23 060	4	85	55
Wisconsin.........	Merrill, Wis.............	2 630	14 907	18 140	6	21	27
Wapsipinicon	Stone City, Iowa.........	1 310	6 395	8 690	6	17.3	20
				8 190		19.7	
Iowa.............	Iowa City, Iowa..........	3 320	12 067	19 450	4	20.0	18.3
Cedar............	Cedar Rapids, Iowa.......	6 320	31 093	52 450	6	29.0	28
Des Moines.......	Keosauqua, Iowa	14 300	61 747	97 140	4	31	29
Illinois..........	Peoria, Ill..............	13 200	41 060	57 650	4	19.4	21

which would normally occur in very long periods, and others may be floods on rivers having coefficients greater than 100, which would normally occur in shorter periods. When it is considered that the formula was obtained from data independent of these floods, the agreement is noticeably close, and serves as a valuable check on the formulas here proposed.

Comparison with Flows in Sewers.—In Table 28 is given a comparison of the proposed formula with that of McMath for obtaining the flow in sewers, $Q = C R \sqrt[5]{S A^4}$. In making the comparison, a period of 20 years is assumed, as it seems that sewers liberally designed should flow at full capacity at least once during such a period. For the proposed formula, C is taken as 65, which is the

TABLE 20.—Values for Coefficient, C, in Formula, $Q = C\ A^{0.8}\ (1 + 0.8 \log. T)$.

Missouri River Basin.

Name of stream. (1)	Location of station. (2)	Catchment area, in square miles, A. (3)	Flood Flows, in Cubic Feet per Second. 24-Hour Average. Average yearly, Q (Ave.). (4)	Flood Flows, in Cubic Feet per Second. 24-Hour Average. Larger flood, Q. (5)	Period of observation. T. (6)	Values of C. From larger floods. (7)	Values of C. From average yearly flood. (8)
Jefferson (continuation of Redrock and Beaverhead)	Sappington, Mont	8 984	9 860	15 470 10 040	9	6.4 6.0	6.7
Missouri	Townsend, Mont	14 500	28 050	52 500 32 500	8	14.3 13.5	13.2
Missouri	Cascade, Mont	18 300	23 524	49 300 38 000	9	10.9 11.2	9.1
Madison	Red Bluff, Mont	2 085	6 500	10 275 8 613 8 325	13	12.1 12.7 13.2	14.4
West Gallatin (head of Gallatin)	Salesville, Mont	860	5 800	10 750 9 700 7 810	15	25 28 27	26
Gallatin	Logan, Mont	1 805	4 860	6 550 6 460	9	9.2 10.7	12.1
Middle Creek	Bozeman, Mont	26	456	806	5	38	34
Ford Creek	Augusta, Mont	18	403	720	4	42	37
Smith Creek	Augusta, Mont	26	587	806	3	43	43
Marias	Shelby, Mont	2 610	11 530	29 500	6	34	21
Milk	Havre, Mont	7 300	3 919	9 600 8 960	13	4.1 4.5	3.2
Milk	Malta, Mont	14 000	7 040	11 200	6	3.3	3.4
Yellowstone	Livingston, Mont	3 580	19 330	26 820 26 525	9	22 25	27
Yellowstone	Glendive, Mont	66 100	69 275	90 600 86 400	8	7.3 8.3	9.7
Boulder (West Fork)	Bruffeys, Mont	94	1 528	1 610	5	27	40
Clark Fork	Fromberg, Mont	2 500	8 200	10 800	4	13.9	15.6
Pryor Creek	Huntley, Mont	800	597	1 370	5	4.2	2.4
Bighorn	Thermopolis, Wyo	8 184	13 900	17 610	6	8.0	10.3
Bighorn	Hardin, Mont	20 700	28 207	40 800	7	8.5	9.9
Shoshone	Cody, Wyo	1 480	11 330	15 800	7	27	33
Shoshone (South Fork)	Marquette, Wyo	500	3 583	5 300	5	24	25
Clear Creek	Buffalo, Wyo	118	710	853	6	11.6	15.6
Little Muddy	Williston, N. Dak	800	1 656	4 340	5	13.2	7.9
Little Missouri	Medora, N. Dak	5 780	9 588	19 000	6	11.6	9.4
Knife	Broncho, N. Dak	1 260	1 940	3 500	7	6.9	6.4
Heart	Richardton, N. Dak	1 250	3 426	8 020	8	15.5	11.4
Cannon Ball	Stevenson, N. Dak	3 650	3 480	5 900	6	5.1	4.9
Cheyenne	Edgemont, S. Dak	7 350	7 600	10 960	4	6.0	6.1
Rapid Creek	Rapid, S. Dak	410	600	880	4	4.9	4.9
Belle Fourche	Belle Fourche, S. Dak	3 250	4 050	5 941	4	6.2	6.3
Niobrara	Valentine (Fort Niobrara), Nebr	6 070	3 749	7 000	5	3.4	3.5
North Platte	Pathfinder, Wyo	12 000	10 800	12 800	4	4.7	5.9
North Platte	Orin Junction, Wyo	14 828	17 520	22 960	4	7.6	8.1
North Platte	Guernsey, Wyo	16 200	14 500	30 000 15 200	8	7.5 6.5	6.2
North Platte	Mitchell, Nebr	24 400	14 227	23 000 23 000	9	4.1 4.7	3.8
North Platte	Camp Clark, Nebr	24 830	16 560	23 560	4	4.9	5.1

TABLE 20.—(*Continued.*)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
North Platte	Bridgeport, Nebr.	23 200	13 258	19 700	5	4.0	4.3
North Platte	North Platte, Nebr.	28 500	17 640	25 500	13	3.7	4.8
				23 720		4.1	
				23 010		4.3	
Platte	Lexington, Nebr.	53 300	19 000	30 000	5	3.1	3.1
Platte	Columbus, Nebr.	56 900	25 000	51 100	14	4.2	3.9
				35 400		4.1	
				30 200		4.0	
Laramie	Woods Landing, Wyo.	435	3 130	4 502	5	22	24.3
South Platte (South Fork)	South Platte, Colo.	2 160	1 129	1 800	6	2.4	2.4
South Platte (South Fork)	South Platte, Colo.	2 610	1 454	2 540	9	2.7	2.7
				2 170		2.9	
South Platte (South Fork)	Denver, Colo.	3 840	1 900	5 570	11	4.1	2.6
				2 425		3.4	
				2 308		3.2	
South Platte	Hersey, Colo.	9 470	4 150	9 335	7	3.7	2.3
South Platte	Orchard, Colo.	12 260	5 875	11 159	4	4.1	3.2
South Platte	Julesburg, Colo.	20 600	5 134	12 900	8	2.6	1.8
Bear Creek	Morrison, Colo.	170	370	737	6	7.5	6.1
Bear Creek	Forkscreek, Colo.	345	1 291	2 260	12	11.4	12.1
				1 994		12.3	
St. Vrains Creek	Lyons, Colo.	209	982	1 280	10	9.9	
				1 145		10.8	13.8
Boulder Creek	Boulder, Colo.	179	720	840	9	7.5	11.3
				826		8.6	
South Boulder	Marshall, Colo.	125	580	1 090	7	13.7	12.2
Big Thompson Creek	Loveland, Colo.	305	1 120	2 090	9	12.2	11.5
				1 685		13.8	
Cache la Poudre	Fort Collins, Colo.	1 060	3 133	5 611	12	11.5	11.9
				5 100		12.5	
Loupe	Columbus, Nebr	13 500	14 940	27 000	12	7.2	7.4
				25 800		8.1	
North Loupe	St. Paul, Nebr.	4 024	5 080	7 500	5	6.3	6.6
Elkhorn	Norfolk, Nebr.	2 474	3 040	8 000	7	9.2	5.9
Elkhorn	Arlington, Nebr.	5 980	6 300	9 568	5	5.8	6.0
Republican	Bostwick, Nebr.	23 300	8 038	24 500	7	4.7	2.6
Republican	Superior, Nebr.	22 347	6 450	14 100	7	2.8	2.1
Republican	Junction, Kans.	25 840	20 650	47 520	11	7.8	6.1
				37 500		7.9	
Kansas	Lecompton, Kans., and Lawrence, Kans.	58 550	59 300	221 000	60 15	13.9 17.5	9.1
		and		130 000		15.8	
				81 400		14.2	
		59 841		67 000		13.2	
Saline	Saline, Kans.	3 311	4 200	7 895	7	7.1	6.4
Solomon	Niles, Kans.	6 815	5 400	10 602	7	5.4	4.6
Blue	Manhantan, Kans.	9 490	27 500	68 770	11	25	18.1
				43 430		23	
Gasconade	Arlington, Mo.	2 720	27 327	44 960	4	54	49

average coefficient for eastern rivers. For the McMath formula, values are assumed as follows: $C = 0.50$, $R = 2.75$ in.; $S = 10$ to 30. These values seem reasonable when it is considered that, during a period of 20 years, an unusually heavy rainfall would probably come on frozen or well-saturated ground, for which the percentage of run-off would be as great as, or greater than, it would be from a 50% impervious area in a city. Table 28 indicates that the flows for small catchment areas, as obtained by the proposed formula, are fairly consistent with sewer experience.

TABLE 21.—Values for Coefficient, C, in Formula, $Q = C\ A^{0.8}\ (1 + 0.8 \log. T)$.

Lower Mississippi River Basin.

Name of stream.	Location of station.	Catchment area, in square miles, A.	Flood Flows, in Cubic Feet per Second. 24-Hour Average. Average yearly. Q (Ave.).	Flood Flows, in Cubic Feet per Second. 24-Hour Average. Larger flood. Q.	Period of observation. T.	Values of C. From larger floods.	Values of C. From average yearly flood.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Arkansas.........	Salida, Colo........	1 160	2 808	3 900	7	8.2	9.9
Arkansas.........	Canon City, Colo..	3 060	3 757	6 690	27	5.1	6.0
				5 611		5.3	
				5 400		5.5	
				5 120		5.6	
				4 925		5.7	
Arkansas.........	Pueblo, Colo.......	4 600	5 430	11 060	19	6.4	6.7
				8 321		6.3	
				7 659		6.5	
				6 980		6.5	
Arkansas.........	Nepesta, Colo.....	9 130	8 996	15 075	5	6.6	6.1
Arkansas.........	Hutchinson, Kans.	34 000	7 296	11 645	6	1.7	1.7
Purgatory.........	Trinidad, Colo.....	742	2 573	4 600	5	14.9	13
Verdigris.........	Liberty, Kans......	3 067	31 881	41 450	8	37	52
				36 950		38	
Neosho...........	Iola, Kans.........	3 670	31 429	45 560	7	38	44
				39 120		42	

TABLE 22.—Values for Coefficient, C, in Formula, $Q = C\ A^{0.8}\ (1 + 0.8 \log. T)$.

Western Gulf of Mexico.

Name of stream.	Location of station.	Catchment area, in square miles, A.	Flood Flows, in Cubic Feet per Second. 24-Hour Average. Average yearly. Q (Ave.).	Flood Flows, in Cubic Feet per Second. 24-Hour Average. Larger flood. Q.	Period of observation. T.	Values of C. From larger floods.	Values of C. From average yearly flood.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Brazos.............	Waco, Tex.........	30 800	55 000	132 000	11	18.4	14.1
				99 000		18.5	
Colorado..........	Austin, Tex.......	37 000	43 000	72 600	10	8.9	9.5
				70 300		10.1	
Rio Grande........	Del Norte, Colo...	1 400	4 850	7 670	17	11.8	13.2
				6 870		12.7	
				5 930		13.0	
				5 650		13.3	
Conejos...........	Mogote, Colo......	282	2 800	4 170	5	29	31

TABLE 23.—VALUES FOR COEFFICIENT, C, IN FORMULA, $Q = C\ A^{0.8}\ (1 + 0.8 \log. T)$.

COLORADO RIVER BASIN.

Name of stream.	Location of station.	Catchment area, in square miles, A.	Flood Flows, in Cubic Feet per Second. 24-Hour Average. Average yearly. $\dot{Q}$ (Ave.).	Larger flood. Q.	Period of observation. T.	Values of C. From larger floods.	From average yearly flood.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Green (head of Colorado).	Green River, Wyo.......	7 450	14 100	21 384	9	9.7	11.3
				17 860		10.3	
Green....................	Greenriver, Utah........	38 200	42 200	62 400	6	8.3	9.1
Colorado	Yuma, Ariz.............	225 000	90 750	149 500	9	4.4	4.7
				116 000		4.5	
Yampa....................	Craig, Colo.	1 730	8 740	9 680	4	16.8	22
White	Meeker, Colo............	634	3 162	3 710	5	13.6	18.1
Duchesne.................	Myton, Utah	2 750	5 550	9 560	9	9.5	9.7
				7 320		9.7	
Lake Fork................	Myton, Utah	475	2 250	3 000	7	12.9	16.3
Uinta....................	Whiterocks, Utah.......	218	1 050	1 430	5	12.4	14.2
Uinta....................	Fort Duchesne, Utah ...	672	2 850	4 520	8	14.3	15.5
				4 470		16.5	
Whiterocks...............	Whiterocks, Utah.......	114	676	1 146	5	16.6	15.4
Grand	Kremmling, Colo........	2 380	10 750	15 300	6	18.9	21
Grand	Glenwood Springs, Colo.	4 520	20 500	27 710	8	19.2	24
				27 600		22.3	
Grand	Palisades, Colo.	8 550	26 725	37 000	7	15.7	19
Frazier..................	Granby (Coulter), Colo..	220	1 725	1 860	4	16.7	23
Williams Fork	Sulphur Springs, Colo...	200	1 077	1 410	5	13.1	15.6
Blue.....................	Kremmling, Colo......	700	5 200	6 020	4	22	27
Eagle....................	Gypsum, Colo...........	800	4 700	6 040	5	18	22
Uncompahgre..............	Montrose, Colo.	565	1 630	2 070	5	8.4	10.2
Dolores..................	Dolores, Colo..........	524	2 035	2 944	8	11.4	13.5
				2 825		13.0	
Animas...................	Durango, Colo	812	4 700	7 800	7	22	22
				5 870		22	
Animas...................	Aztec, N. Mex..........	1 300	6 510	12 500	5	26	21
Gila.....................	San Carlos, Ariz.........	13 460	5 947	8 500	6	2.5	3.0
Salt.....................	McDowell, Ariz.........	6 260	43 800	138 000	6	78	40
Verde	McDowell, Ariz.........	6 000	47 740	60 000	4	39	46

By grouping rivers according to their values of C, and investigating the rainfall conditions, storage, slopes, amount of sandy area, etc., for these different groups, it seems probable that much valuable information could be obtained with the data now available.

The writer feels that the method of rating and comparing rivers by using the average yearly floods affords a means of obtaining valuable information on many rivers, with a comparatively small amount of labor, and hopes that in the future the maximum flood for each year will be observed for many more rivers. It is also to be hoped that

TABLE 24.—Values for Coefficient, C, in Formula, $Q = C\,A^{0.8}\ (1 + 0.8 \log. T)$.

Great Basin.

Name of stream. (1)	Location of station. (2)	Catchment area, in square miles, A. (3)	Flood Flows, in Cubic Feet per Second. 24 Hour Average. Average yearly, Q (Ave.). (4)	Larger flood, Q. (5)	Period of observation, T. (6)	Values of C. From larger floods. (7)	From average yearly flood. (8)
Bear	Dingle, Idaho	2 890	2 670	4 050 3 990	7	4.1 4.8	4.5
Bear	Preston, Idaho	4 500	4 580	8 500 7 900 6 380 6 100	20	5.1 5.5 5.5 5.6	5.5
Bear	Collinston, Utah	6 000	6 550	11 600 10 590 10 200 82 200	21	5.3 5.7 6.1 6.1	6.1
Logan	Logan, Utah	218	1 390	2 450 1 930 1 823	12	17.8 18.3 18.9	18.8
Black Fork	Hyrum, Utah	286	670	1 900 956	9	11.7 10.2	7.3
Weber	Oakley, Utah	163	2 730	4 010	6	50	55
Weber	Devils Slide, Utah	1 090	4 300	5 880	4	14.1	16.0
Weber	Plain City, Utah	2 060	5 030	7 580	4	11.3	11.2
Weber	Uinta, Utah	1 600	4 800	7 980 7 230	10	12.6 13.3	13.1
American Fork	American Fork, Utah	66	400	885	5	20	14.3
Big Cottonwood	Salt Lake City, Utah	48.5	460	835 793	11	21 23	21
Mill Creek	Salt Lake City, Utah	21.3	56	112 72	12	5.2 4.9	4.8
Parley's Creek	Salt Lake City, Utah	50.1	142	274 228	12	6.4 6.8	6.3
Emigration Creek	Salt Lake City, Utah	29	18	24 22	8	0.9 1.0	1.2
City Creek	Salt Lake City, Utah	19.2	82	164 132	11	8.5 8.8	7.7
Provo	Provo, Utah	640	2 130	4 150 3 620 3 310 2 600	18	11.8 12.5 12.9 12.7	12.1
Hobble Creek	Springville, Utah	120	334	820	6	11.0	7.2
Spanish Fork	Spanish Fork, Utah	670	860	1 970	9	6.1	4.6
Sevier	Marysvale, Utah	2 560	1 370	3 000	6	3.5	2.6
Sevier	Gunnison, Utah	3 990	1 095	2 240	9	1.6	1.4
Sevier	Leamington, Utah	5 595	1 560	2 329	4	1.6	1.5
San Pitch	Gunnison, Utah	836	210	338	5	1.0	1.0
Humboldt	Golconda, Nev	10 800	1 400	3 160 3 100 3 080	15	0.9 1.0 1.1	0.8
Humboldt	Oreana, Nev	13 800	1 260	3 047 2 616	14	0.8 0.8	0.6
Humboldt (North Fork)	Halleck, Nev	1 020	530	1 020	6	2.5	2.0
Humboldt (South Fork)	Elko, Nev	1 150	1 120	1 478 1 400 1 385	13	2.8 3.1 3.3	4.0
Humboldt	Elko, Nev	2 840	1 650	2 396	7	2.5	2.8
Humboldt	Palisade, Nev	5 010	1 834	2 620	4	1.9	2.0

TABLE 24.—(*Continued.*)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Truckee	Tahoe, Cal	519	774	1 340	10	5.0	5.2
				931		4.9	
Truckee	State Line, Col.-Nev	955	5 260	15 300	10	35	22
				8 110		31	
Truckee	Vista, Nev	1 520	4 930	8 940	10	14.2	14.1
				7 510		15.0	
Walker (West Fork)	Coleville, Cal	306	2 300	4 170	6	26	24
Walker	Yerington, Nev	1 100	700	1 700	7	3.8	2.6
Donner Creek	Truckee, Cal	30	560	980	8	38	37
				862		41	
Prosser Creek	Hobart Mills, Cal	48	518	924	5	27	24
Little Truckee	Starr, Cal	166	1 370	1 809	4	20	23
Independence Creek	Independence Lake, Cal	8.5	230	286	5	33	42
Carson (East Fork)	Gardnerville, Nev	361	2 400	3 430	8	18	22
				3 162		20	
Carson	Empire, Nev	988	2 561	4 000	9	9.2	10.3
				3 791		10.3	
Carson (West Fork)	Woodward, Cal	70	900	1 570	12	28	30
				1 450		31	
Carson (East Fork)	Rodenbohs, Nev	414	3 700	5 540	4	30	29
Susan	Susanville, Cal	256	1 300	1 800	4	14.5	15
Silvies	Burns, Ore	865	2 260	4 730	5	13.6	10
Chewaucan	Paisley, Ore	272	1 580	3 500	5	25	18
Ogden	Ogden, Utah	360	1 690	3 257	11	16.0	15.2
				2 433		16.2	

records of both the 24-hour average rate and the maximum rate of flow will be made for our larger floods, and that for all records of floods it will be clearly stated as to which rate it refers. In this study it was difficult, in many cases, to distinguish between the two.

Acknowledgments.—As far as the writer knows, the first suggestion of the use of the element of time in a formula for flood flows was made by Allen Hazen, M. Am. Soc. C. E., in a memorandum written by him in 1910. This memorandum contained a study of maximum floods on some eastern rivers, and suggested a formula of a nature similar to the one proposed in this paper. Mr. Hazen has followed closely the progress of the study which is here presented, and to him the writer is indebted for many valuable suggestions.

The late Richard Hazen, Jun. Am. Soc. C. E., was associated with the writer in the earlier and essential portions of this study, and it was the intention that he should write the paper jointly with the writer. As this is now impossible, the writer can but acknowledge his indebtedness and pay tribute to the great amount of able work done by Mr. Richard Hazen in the preparation of this paper.

The writer also wishes to acknowledge his indebtedness to those who have furnished data for the paper. Free use has been made of

TABLE 25.—VALUES FOR COEFFICIENT, C, IN FORMULA, $Q = C\,A^{0.8}\,(1 + 0.8 \log. T)$.

SOUTHERN PACIFIC COAST.

Name of stream.	Location of station.	Catchment area, in square miles, A.	Flood Flows, in Cubic Feet per Second. 24-Hour Average. Average yearly, Q (Ave.).	Larger flood. Q.	Period of observation. T.	Values of C. From larger floods.	From average yearly flood.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Cottonwood Creek	Jamul, Cal	270	1 940	5 800	4	45	22
San Diego	San Diego (flume near Lakeside), Cal	208	1 900	3 800	4	37	27
Malibu Creek	Calabasas, Cal	97	2 560	6 800	4	117	65
Arroyo	Soledad, Cal	215	3 800	6 250	8	49	51
Mojave	Victorville, Cal	400	4 310	13 413	5	72	36
				4 820		30	
Sacramento	Jellys Ferry (near Red Bluff), Cal	9 300	129 000	254 000	15	87	86
				196 000		88	
				184 600		91	
Pitt	Bieber, Cal	2 950	13 000	27 500	5	30	22
McCloud	Gregory, Cal	608	20 000	41 000	6	149	118
Stony Creek	Fruto, Cal	601	16 200	29 300	9	100	98
				26 500		111	
Feather (North Fork)	Below Prattville, Cal	506	5 750	9 850	4	46	40
Feather	Oroville, Cal	3 640	88 100	187 000	8	154	134
Butte Creek	Butte Valley, Cal	73	1 150	1 640	4	36	37
Indian Creek	Crescent Mills, Cal	740	7 000	11 400	4	39	35
Yuba	Smartville, Cal	1 220	57 000	111 000	6	233	192
Bear	At Vantrent (above Wheatland), Cal	263	17 000	25 800	5	194	193
American	Fairoaks, Cal	1 910	55 500	105 000	5	158	130
Cache Creek	Lower Lake, Cal	500	2 300	4 340	9	17.0	15.8
				3 680		18.2	
Cache Creek	Yolo, Cal	1 230	13 500	20 100	7	40	45
Putah Creek	Winters, Cal	805	22 100	30 000	4	95	103
San Joaquin	Herndon, Cal	1 637	17 500	21 372	7	35	47
				20 780		40	
Kern	Bakersfield, Cal	2 345	4 025	9 505	20	93	80
				8 851		102	
				5 384		96	
Tule	Portersville, Cal	266	3 100	5 430	8	36	35
Kaweah	Threerivers, Cal	520	4 560	9 210	6	38	30
Kings	Sanger, Cal	1 740	19 000	43 930	14	58	49
				26 600		54	
Merced	Above Merced Falls, Cal	1 090	13 000	27 500	8	59	48
Tuolumne	La Grange, Cal	1 500	18 900	52 000	15	76	54
				24 400		64	
				21 800		60	
Stanislaus	Oakdale, Cal	1 051	9 700	13 940	5	34	37
Stanislaus	Knights Ferry, Cal	935	25 000	57 200	6	150	106
Mokelumne	Clements, Cal	642	8 530	15 300	5	55	49

many papers and reports. Records of great value, owing to their completeness and reliability, have been furnished by John H. Cook, M. Am. Soc. C. E., for the Passaic River; by Richard A. Hale, M. Am. Soc. C. E., for the Merrimac River; and by James L. Tighe, M. Am. Soc. C. E., for the Fomer River.

TABLE 26.—VALUES FOR COEFFICIENT, C, IN FORMULA, $Q = C\ A^{0.8}\ (1 + 0.8 \log. T)$.

NORTHERN PACIFIC COAST.

Name of stream.	Location of station.	Catchment area, in square miles, A.	Flood Flows, in Cubic Feet per Second. 24-Hour Average. Average yearly. Q (Ave.).	Larger flood. Q.	Period of observation. T.	Values of C. From larger floods.	From average yearly flood.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Rogue	Tolo, Ore.	2 020	34 325	48 300	4	74	78
Umpqua (South Fork)	Brockway, Ore.	1 800	49 725	70 700	4	119	124
Columbia	Cascade Locks and The Dalles, Ore.	237 000	754 100	1 390 000	31	32	38
				1 133 000		29	
				1 040 000		30	
				903 000		30	
Clark Fork	Newport, Wash.	24 000	99 100	155 000	7	29	31
Missoula	Missoula, Mont.	5 960	20 544	35 800	7	20.0	19
Bitterroot	Grantsdale, Mont.	1 550	9 487	12 875	6	22	27
Bitterroot	Missoula, Mont.	3 260	23 070	37 437	4	39	35
Spokane	Spokane, Wash.	4 000	23 550	35 200	18	23	31
				32 875		25	
				31 500		27	
				29 024		27	
Salmon Creek	Malott, Wash.	152	356	577	5	6.7	6.5
Methow	Pateros, Wash.	1 710	11 307	11 960	6	19	29
Chelan	Chelan, Wash.	950	8 540	9 800	6	25	35
Wenachee	Cashmere, Wash.	1 250	16 450	19 000	4	43	55
Yakima	Martin, Wash.	56	2 640	6 150	6	151	105
Yakima	Cle Elum, Wash.	500	15 310	25 600	4	119	105
Yakima	Umtanum, Wash.	1 540	25 000	41 000	4	78	70
Yakima	Union Gap (near Yakima, Wash.)	3 300	29 280	63 900	7	58	45
Kachess Lake, on Kachess River	Easton, Wash.	63	1 530	2 300	4	56	55
Clealum Lake	Roslyn, Wash.	205	7 500	17 700	6	154	106
Naches	Nile, Wash.	63.6	8 412	21 100	6	74	48
Naches	North Yakima, Wash.	1 120	11 948	21 900	7	47	44
Snake (South Fork)	Moran, Wyo.	820	9 146	20 890	7	58	43
Snake (South Fork)	Lyon, Idaho	5 480	33 457	51 450	7	31	34
Snake (South Fork)	Near Minidoka, Idaho	17 900	28 835	38 000	7	9.0	11.4
Snake (South Fork)	At Minidoka, Idaho	22 600	41 540	53 100	4	11.8	13.7
Snake (North Fork)	Ora, Idaho	1 040	3 800	5 370	7	12.2	14.6
Fall	Fremont, Idaho	390	2 984	4 160	5	23	25
Teton	Near St. Anthony, Idaho	960	4 390	7 620	7	18.4	18
Blackfoot	Presto, Idaho	1 020	1 870	2 370	5	6.0	7.3
Boise	Highland, Idaho	2 610	11 716	17 000	5	20	22
Boise	Boise, Idaho	2 450	17 880	40 130	9	44	34
				28 572		43	
Malheur	Vale, Ore.	4 190	7 893	14 540	5	11.8	10
Malheur	Near Vale, Ore.	9 900	3 571	4 445	4	1.9	2.2
Payette	Horse Shoe Bend, Idaho	2 240	14 237	19 500	4	27	29
Weiser	Weiser, Idaho	1 670	9 732	17 940	12	26	26
				17 115		28	
Grande Ronde	Hilgard, Ore.	660	3 225	4 607	4	17.3	18
Grande Ronde	Elgin, Ore.	1 350	5 696	8 349	5	16.8	18
Wallowa	Joseph, Ore.	47	659	728	4	22	30
Palouse	Hooper, Wash.	2 210	6 526	17 200	9	21	13.7
				16 430		23	
Owyhee	Owyhee, Ore.	11 100	12 250	20 920	9	6.9	7.1
				18 000		7.4	

TABLE 26.—(*Continued.*)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Umatilla	Gibbon, Ore.	353	3 808	10 000	10	52	35
				4 217		42	
Umatilla	Yoakum, Ore.	1 200	9 200	23 900	6	51	32
John Day	McDonald, Ore.	7 800	13 070	22 800	5	11.1	10
Deschutes	Allen Ranch (near Lava), Ore.	880	1 320	2 150	4	6.4	6
Deschutes	At West's Ranch (near Lava), Ore.	1 240	2 500	4 000	4	9.2	8.5
Deschutes	Biggs, Ore.	9 180	18 050	30 600	4	14.3	12
Willamette Middle Fork)	Jasper, Ore.	1 450	62 850	122 000	4	244	186
Willamette	Albany, Ore.	4 860	115 500	188 000	19	102	130
				182 000		122	
				179 000		126	
				168 000		130	
Willamette (Coast Fork)	Goshen, Ore.	690	20 500	31 300	4	109	109
						113	
McKenzie	Springfield, Ore.	960	27 475	37 900	4	106	113
Yamhill	Sheridan, Ore.	290	15 475	18 100	4	133	165
Cedar	Ravensdale, Wash.	170	4 612	10 800	10	98	76
				6 420		91	

From the report of Emil Kuichling, M. Am. Soc. C. E., on the Water Supply for the New York State Barge Canal, many data, for both American and foreign streams, have been taken, as well as a summary of the formulas previously proposed.

TABLE 27.—MISCELLANEOUS MAXIMUM FLOODS, ARRANGED IN ORDER OF VALUES OF K.

$$K = \frac{Q\ (\text{Max.})}{A^{0.8}\,(1 + 2\,A^{-0.3})}.$$

Stream. (1)	Area. (2)	Date. (3)	Flood. (4)	$1+2A^{-0.3}$ (5)	$A^{0.8}$ (6)	K (7)
Dry Run, Ohio	9.8	1912	4 500	1.98	5.3	360
Westfield, Mass	356.0	1878	53 000	1.34	110.0	358
Esopus, at Saugerties, N. Y	417.0	......	55 000	1.32	125.0	333
Burgoons Run, Pa	8.0	1889	3 200	2.07	5.28	292
Lake Roland, Md	39.0	1868	9 000	1.67	18.7	288
Br. Conemaugh, at Johnstown, Pa. (Johnstown flood)	48.6	1889	10 000	1.63	22.4	275
Sawkill, N. Y., 45 miles above mouth	35.0	1895	8 000	1.69	17.2	275
Chemung, at Elmira, N. Y	2 055.0	1889	138 000	1.20	446	257
Neshaminy, below Forks, Pa	139.0	......	19 000	1.45	52	252
Little Tennessee, at Judson, Tenn	675.0	......	57 500	1.28	184	245
Six Mile Creek, at Ithaca, N. Y	46.0	1905	8 500	1.63	21.4	243
Potomac, at Great Falls, Md	11 500.0	1889	470 000	1.12	1 750	240
East Branch, Delaware	920.0	1904	72 000	1.26	238	240
Tohickon, at Pt. Pleasant, Pa	102.0	......	14 100	1.50	40.5	232
Tennessee, at Chattanooga, Tenn	21 418.0	1867	735 000	1.10	2 910	228
Savannah, at Augusta, Ga	7 300.0	......	310 000	1.14	1 230	221
Perkiomen, at Frederick, Pa	152.0	......	17 600	1.44	56	217
Spring Creek, Pa	11.6	1908	3 000	1.96	7.10	216
Croton, at Croton Dam, N. Y	339.0	1867	30 000	1.35	105.0	212
Catskill, at Woodstock, N. Y	210.0	1901	21 000	1.40	72	208
Susquehanna, at Harrisburg, Pa	24 030.0	1889	700 000	1.10	3 200	200
Ramapo, N. J	118.0	1903	12 500	1.49	42.3	198
Rock Creek, D. C	77.5	......	9 800	1.54	32.7	195
Gunpowder, Md	302.0	1889	25 000	1.36	96.4	191
Millbrook, N. Y	9.4	1905	2 300	2.02	6.10	191
Delaware, at Stockton, N. J	6 855.0	1841	255 000	1.14	1 170	190
Monongahela, Pa	5 430	1888	207 000	1.15	975	184
Nine Mile Ck., at Stittville, N. Y	62.6	......	7 820	1.58	27.5	180
Raritan, at Bound Brook, N. J	879	1882	52 000	1.26	230	180
Nashua, Mass	109.0	1848	11 400	1.49	42.8	179
Stony Creek, at Johnstown, Pa	428	......	30 000	1.32	127	179
Youghiogheny, Pa	782.0	1888	46 000	1.27	206	177
Flat River, R. I	61.0	1843	7 350	1.58	26.5	176
Susquehanna, at McCalls Ferry	26 766.0	1904	671 000	1.09	3 500	176
Trout Brook, at Brooksport, N. Y	25.0	......	3 950	1.76	13.1	171
Pequonnock, Conn	25	......	4 000	1.76	13.2	171
Kennebec, Me	4 270	1901	156 800	1.16	802	168
Schoharie, N. Y	930	1901	49 600	1.26	236	167
Tuckaseegee, at Bryson, N. C	662	......	38 750	1.28	180	163
Mad Brook, N. Y	5	1905	1 300	2.23	3.62	161
Susquehanna, at Danville, Pa	11 070	1902	305 000	1.12	1 720	159
Fomer, above reservoir, Mass	13	......	2 880	1.92	7.8	159
Buffalo Creek, N. Y	420	1902	23 000	1.32	125	152
Ohio, Pa	19 000	1907	440 000	1.10	2 650	151
Rhine, at Macon. Ga	2 574	......	96 500	1.19	535	151

TABLE 28.

Catchment area, in square miles. (1)	FLOW FROM FORMULA: Q (Max.) $= C\,A^{0.8}\,(1 + 0.8 \log. T)\,(1 + 2\,A^{-0.3})$ $C = 65$, $T = 20$; A in square miles. (2)	FLOW BY MCMATH FORMULA: $Q = C\,R\,\sqrt[5]{S\,A^4}$ $C = 0.50$; $R = 2.75$ in.; A in acres. (3)
0.1	104	76 ($S = 30$)
1.0	398	440 ($S = 20$)
10.0	1 680	2 420 ($S = 10$)

TABLE 29.—FORMULAS WHICH HAVE BEEN PROPOSED FOR USE IN OBTAINING THE MAXIMUM FLOOD FLOW.
The following formulas are taken from the Report on the Barge Canal, State of New York, 1901.
Part 14 of the Report on Water Supply by Emil Kuichling, pp. 844 to 851.

No. (1)	Author of formula. (2)	Formula. (3)	Notes. (4)
1	Fanning	$Q = 200\ A^{\frac{5}{6}}$	Adapted to rivers in New England, etc. A = catchment area, in square miles.
4	Dickens	$Q = 500\ A^{\frac{3}{4}}$	Adapted to rivers in Central Provinces, India. This formula is also given with the following coefficients: 200, 825, 1 200, and 2 200; each one is applicable to a different part of India; 500 is here taken as a representative value.
8	Ganguillet	$Q = \dfrac{1\,421\ A}{3.11 + \sqrt{A}}$	Adapted to Swiss streams.
9	Italian	$Q = \dfrac{1\,819\ A}{0.311 + \sqrt{A}}$	Adapted to streams in Northern Italy.
10	Italian	$Q = \dfrac{2\,600\ A}{0.311 + \sqrt{A}}$	Adapted to small brooks in Northern Italy.
11	O'Connell	$Q = \sqrt{458\ (640\ A + 4.58)} - 45.8$	Adapted to small districts.
12	Dredge or Burge	$Q = 1\,300\ \dfrac{A}{L^{\frac{2}{3}}}$	If A is taken as a rectangle having dimensions L and $\frac{L}{2}$, $A = \frac{L^2}{2}$, and the formula reduces to $Q = 1\,030\ A^{\frac{2}{3}}$.
13	Craig	$Q = 440\ n\ B$ hyp. log. $\dfrac{8\ L^2}{B}$	n = coefficient for the drainage basin, assumed as 1.16; L = extreme length of drainage basin; B = average width of drainage basin. If L is taken as being equal to $2\ B$, the formula reduces to $Q = 365\ \sqrt{A}$ hyp. log. $(22.7\sqrt{A})$.
14	Bürkli-Ziegler	$Q = C_1\ R_1 \left(\dfrac{S_1}{A}\right)^{\frac{1}{4}} A$	Assume for $C_1 = 70$ $R_1 = 3$ in. of rain per hour. $S_1 = 4$ (slope per 1 000) Formula reduces to $Q = 296\ A^{\frac{3}{4}}$.

TABLE 29.—(*Continued.*)

No. (1)	Author of formula. (2)	Formula. (3)	Notes. (4)
15	Possenti	$Q = C_2 \frac{R_2}{L_1}\left(A_2 + \frac{A_1}{3}\right)$	L = length of water-shed. Assume for $C_2 = 1\,010$ (Kuichling), $R_2 =$ 6 in. rain in 24 hours, $L_1 = \sqrt{2A}$ as in (12), $A_2 = \frac{A}{2}$ Hilly part of water-shed, $A_1 = \frac{A}{2}$ Level part of water-shed, Formula reduced to $Q = 2\,856\, A^{\frac{1}{2}}$.
16	Cramer	$Q = \frac{C_3 R_3 m\ A\ (S_2)^{\frac{1}{3}}}{9 + (0.0658\ m\ R_3\ A)^{\frac{1}{2}}}$	Reduced by Kuichling, for conditions on Mohawk River, to $Q = \frac{80.6}{1 + 0.1\,347\,(A)^{\frac{1}{2}}} A$
17	Lauterburg	$Q = A\left[\frac{615}{6 + 0.00259\,A} + 0.53\right]$	
	Kuichling	$Q = \left(\frac{44\,000}{A + 170} + 20\right) A$	(1) for flood exceeded occasionally.
		$Q = \left(\frac{127\,000}{A + 370} + 7.4\right) A$	(2) for flood exceeded rarely.
	E. C. Murphy and others	$Q = \left[\frac{46\,790}{A + 320} + 15\right] A$	Formula taken from U. S. Geological Survey, Water Supply and Irrigation Paper No. 147.

TABLE 30.—FLOOD FLOWS ON FOREIGN STREAMS.

(Data from Report on Barge Canal, State of New York, Part 14, by Emil Kuichling.)

$$K = \frac{Q}{(A^{0.8})\,(1 + 2\,A^{-0.3})}.$$

River. (1)	Area. (2)	Date (3)	Flood. (4)	$1 + 2A^{-0.3}$ (5)	$A^{0.8}$ (6)	K (7)
Pliessnitz, Germany	58.0	1890	47 700	1.59	25.5	1 180
Irrity, India	336	....	150 000	1.34	104	1 070
Towbrapoorny, India	587	....	190 000	1.29	165	890
Dept. de l'Ardeche, France	938	1827	247 000	1.25	239	827
Zacken, Germany	42.5	....	21 000	1.65	20.0	635
Kemlitz and Dittelsdorfwasser, Germany	7.4	1887	6 790	2.11	5.1	630
Spree, Germany	6.7	1887	6 210	2.12	4.70	620
Wittgendorfbach, Germany	3.6	1887	3 650	2.37	2.80	550
Landwasser, Germany	6.1	1887	5 050	2.16	4.30	540
Kemlitz, Germany	5.4	....	4 500	2.20	3.90	515
Dittelsdorfwasser, Germany	2.0	1887	2 287	2.61	1.76	495
Landwasser, Germany	3.8	1887	3 400	2.35	2.95	490
Queis, Germany	116.0	1888	30 700	1.48	45.0	460
Tessin, Bellinzona, Italy	541	....	89 000	1.30	154	444
Loire, France	2 200	1846	248 000	1.20	475	435
Wittgendorfbach, Germany	1.3	1887	1 450	2.8	1.25	415
Bruna, Italy	189	....	35 700	1.41	67.0	375
Landwasser, Germany	20.1	1887	7 350	1.81	11.0	370
Stream in Switzerland	87.5	....	18 500	1.52	35.0	350
Nebenwasser, Germany	7.2	1887	3 550	2.11	4.90	340
Queis, Germany	183.5	1888	30 200	1.41	65.0	330
Allaciente, Italy	32.4	....	8 350	1.70	16.0	305
Hungary	7.7	1875	3 170	2.09	5.20	290
Mandau, Germany	50.2	1887	10 500	1.61	23.0	285
Rhine, Germany	1 620	....	121 000	1.22	370	268
Brook near Dublin, Ireland	10.84	1891	3 560	1.98	6.8	260
Woodhead Reservoir, England	11.0	1849	3 520	1.98	6.9	260
Mandau, Germany	119	1890	16 000	1.48	46.0	235
Ostrawitza, Germany	313	1894	34 000	1.35	99	254
Murg, Germany	184	....	22 900	1.41	65	250
Kinzig, Germany	386	....	38 800	1.33	117.0	250
Loire, France	6 945	....	318 500	1.14	1 185	240
Schopsbach, Germany	3.3	1887	1 500	2.40	2.65	235
Murg, Germany	246	....	24 700	1.38	80	224
Wittig, Germany	122	1880	14 800	1.47	47	214
Kinzig, Germany	550	....	42 200	1.30	156	208
Wiese, Germany	163	....	17 600	1.43	59	208
Töss, Germany	68	1876	8 500	1.57	27	200
Olsa, Germany	485	....	33 700	1.32	128	198
Iller, Germany	367	....	27 200	1.34	112	179
Elz, Germany	185	....	15 900	1.41	65	172
Torside and Rhodeswood Reservoir, England	24.1	1852	3 860	1.77	12.8	170
Serein, France	226	....	17 700	1.39	77.0	165
Serein, France	108.3	....	10 600	1.49	43.0	165
Olsa, Germany	291	....	20 800	1.36	94	163
Brenne, France	145	1874	12 400	1.45	54	159
Cure, France	208	....	15 900	1.40	72	159
Lausitzer, Germany	481	1880	29 000	1.31	140	158
Neckar, Germany	4 770	....	159 000	1.16	875	157
Ombrone, Italy	1 620	....	69 500	1.22	870	154
Medlock, England	18.8	1857	3 030	1.83	10.6	156

DISCUSSION

Mr. Morgan.

ARTHUR E. MORGAN, M. AM. SOC. C. E. (by letter).—Mr. Fuller's paper suggests several very profitable fields of inquiry, and is sure to stimulate further investigations that will be of definite value.

One of the fundamental assumptions is that weather conditions throughout the United States are not subject to "permanent changes." It is also assumed that climatic conditions which will produce maximum storms do not occur over the country as a whole during any given year, but that storm conditions are more or less local. An examination of the monthly and yearly precipitations at rainfall-recording stations in all parts of the United States seems to indicate the general accuracy of this assumption for the last 100 years. However, certain notable facts stand out which lead one to question its accuracy for the interior part of the United States.

For instance, in 1844, the largest flood ever recorded occurred in the Red River, through Texas and Arkansas, this river having a drainage area of about 50 000 sq. miles. During the same year the greatest flood on record occurred in the Kaw River, at Kansas City, where the drainage area is 35 000 sq. miles, and the same year produced the greatest flood in a century and a quarter on the Mississippi River, at St. Louis, and the greatest floods on record in the Missouri and Illinois Rivers. The areas over which the floods of 1844 were the greatest on record, are comparable to the total area of France and Germany. Again, in 1903, the greatest flood since 1844 occurred in the Kaw River, at Kansas City, in the Mississippi River, at St. Louis, and in numerous other streams throughout the Central States. As far north as central Minnesota, the heaviest rainfall ever recorded occurred at the same time. Also, in 1903, floods, among the greatest on record for those streams, occurred on the Verde River, in Arizona, the Colorado River, the Raritan and the Delaware, in New Jersey, Cape Fear River, in North Carolina, the Alabama River, in Alabama, the Monongahela River, and on various other streams, from South Carolina to Oregon. Although there is no definite evidence that periods of maximum floods will occur over the entire country at the same time at long intervals, it appears that so little is known about the facts of the case that this assumption is far from being proven for long periods under conditions existing in the central part of the United States. It is possible that certain seasons at long intervals are subject to unusual storm disturbances resulting in floods, though the average yearly or even monthly rainfall may not be above the normal for the country as a whole.

Mr. Fuller assumes that the relation between the average annual flood and the maximum possible flood is the same for largely different

Mr. Morgan.

sections of the country, and the same for the United States as for Europe. He draws attention to the fact that in the arid regions of the United States rainfall and floods are very erratic, and that, though several years may pass without a flood, very large floods may occur.

Although the data are very meager, the writer believes that the climatic conditions which cause this tendency are not limited to the arid regions, but extend to a greater or less degree over the entire United States between the Appalachian and Rocky Mountains, which is the region in which he is particularly acquainted with flood flows.

For instance, careful calculations of flow on the Miami River, at Dayton, Ohio, have recently been made. A gauging station has been maintained in good order at this point for 20 years. The average annual flood for this period is approximately 32 000 cu. ft. per sec. from an area of about 2 500 sq. miles. According to the formula proposed by Mr. Fuller, the maximum flood for a century should be about 2.9 times the average annual flood, or about 85 000 cu. ft. per sec. According to fairly dependable approximations, this flood flow has been largely exceeded four times in the last 107 years. According to Mr. Fuller's formula, the maximum possible flood in 1 000 years would be about 3.9 times the average annual flood, whereas, according to very careful calculations of flow, it appears that the flood of March, 1913, was about 8 times the average annual flood. It may be that climatic conditions show a greater range of variation west of the Appalachian Mountains than east of that range, or in Europe, from which most of the long-time records referred to by Mr. Fuller were secured.

Another assumption by Mr. Fuller is that the same ratio between average annual floods and maximum possible floods exists for streams, regardless of the size of the drainage area. The writer believes that general experience will fail to uphold this assumption. It is known that small drainage areas are subject to very extreme precipitations, and that the most extreme of these precipitations are always local.

For instance, there is the case of the flood on Devil's Creek, in Iowa, which occurred in 1905. Rough estimates of this flow made by the U. S. Geological Survey* indicate that, from an area of 143 sq. miles, the run-off was about 1 300 sec-ft. per sq. mile. From rainfall records at the surrounding stations it appears that from 10 to 12 in. fell on the water-shed, the greater part of it in 12 hours.

Although little dependence can be placed on such estimates of run-off as are recorded for this flood, the rain which fell was sufficient to produce an average 24-hour run-off of probably not less than 200 sec-ft. per sq. mile, or 8 in. from the entire water-shed. This is in a region where the average annual rainfall is about 35 in. and where the annual average maximum rainfall for 24 hours will not exceed about 2 in.,

* Reported in Water Supply and Irrigation Paper No. 162.

Mr. Morgan. the heavier storms coming during the summer when run-off from small rainfalls is light. It is safe to assume that during the flood of 1905 the maximum 24-hour run-off for this territory was not less than 10 times the average annual flood. This extreme case is given simply to illustrate the fact that the range of flood run-off is far greater for small than for large areas.

Similarly, during the flood of March, 1913, in the Miami Valley, certain tributaries with drainage areas of 50 or 100 sq. miles indicate a maximum run-off of at least 400 sec-ft. per sq. mile, which is probably at least ten times the average annual flood. On large water-sheds of 10 000 sq. miles or more, excessive rainfall in one part of the water-shed is usually balanced by lack of rainfall in another part, and the ratio between the average annual flood and the maximum possible flood must be less than for small areas.

When alluvial streams overflow, sand and silt are deposited along the banks up to within 2 or 3 ft. of the surface of the overflow. Along the lower Mississippi and the Red Rivers, in Texas and Arkansas, and the Arkansas River, in Arkansas, there is no evidence of such silt and sand being deposited at elevations very much above those of recent floods. The conclusion is that during recent centuries there have not been floods in which the stage of water was very much above that of recent ones. The only points at which such comparisons can be made safely are along the abandoned loops and cut-offs in the rivers, where the banks have not caved in. On the contrary, on small streams evidence of very different conditions is found. It is fairly common experience, along streams with water-sheds of from 10 to 500 sq. miles, especially in the northern and mid-western States, to find deposits of gravel very evidently placed by flood-water at elevations very much above the elevation of any recorded flood.

The inference is that all the interior part of the United States is subject to very rare local floods of very great intensity, comparable, perhaps, to the one already referred to on Devil's Creek, and that the run-off of such maximum possible floods may be ten times as great as the average annual flood, instead of three or four times as great, as indicated by Mr. Fuller's formula. The suggestion is that, in determining the relation between average annual floods and the maximum possible flood, the extent of the drainage area is an important factor.

The data do not indicate the extent to which flood records have been examined, in the preparation of the paper, but the flood flows on foreign streams outlined in Table 30 would not, it seems, furnish a reasonable basis for the acceptance of such a formula as suggested. For instance, of the floods for which the date is given, more than one-third occurred in 1887 and more than one-half of them in a period of 7 years.

This paper is valuable in that it suggests pointedly that there is a definite relation between the average annual flood on any stream and the maximum possible flood for that stream, and in suggesting that, to a certain extent, short-time records for many streams will serve the same purpose as long-time records for a single stream. Mr. Morgan.

When one considers the number of assumptions which must be made in the use of the formula for determining the maximum possible flood, it would seem that in some cases it might be better to estimate the maximum possible flood by what might be called the rational method, that is, by determining from the basis of experience and the maximum rainfall to be expected, the relation of rainfall to run-off under the conditions which would exist in an assumed case, considering the elements of topography, shape of the drainage basin, direction of storms, season of the year, etc.

There is just one other point that should be mentioned. Mr. Fuller states very clearly that there are two kinds of variables to be met in flood flow: one class varies with different rivers, and includes topography, soil conditions, etc.; the other class of variables, including possibilities of rainfall, weather, temperature, etc., may be alike for many streams, but vary as to time. A statement has been made by the author, which does not appear in the paper; this indicates to the writer the value of his work, but also possibly, the danger of feeling too secure by an improper placing of the burden on those two elements. For instance, if we use his formula, we must assume a value for C, that is, a value representing the variable conditions with regard to water-sheds, etc. If C is made to fit the case, the formula can be made to fit any particular river or flood, but unless we know how that factor, C, was determined, we do not know whether the formula fits any particular case accidentally, or whether C has been determined properly. Mr. Fuller has mentioned one particular river in which he assumed the coefficient, C, to be 100, that is the Miami above Dayton, Ohio. The difference in elevation there between the top of the water-shed and the river valley is seldom more than 200 ft., as compared with mountain streams where the variation may be 1 000 ft.; and the soil formation is variable, being gravelly in some cases. At the time of examining this paper, the writer discussed it with other engineers in his office and came to the conclusion that one could not assume for that valley any higher factor for C than 50. The point is, that in plotting various storms, unless the engineer is thoroughly familiar with the variable factor, C, on one water-shed, as compared with the value of that factor on other water-sheds, he does not know the extent to which the intensity of flood flow is determined by the variable, C, which is constant for that water-shed, and to what extent it is determined by the time variable, including rainfall, season, temperature, etc.

Mr. Hinckley.

H. V. Hinckley, M. Am. Soc. C. E. (by letter).—In addressing the Oklahoma Engineering Society, on December 27th, 1911, the writer said:

"Some of us are perhaps not aware that a 20 or 30-year record of high water, minimum or maximum flow, or rainfall, may often be of comparatively little value. It seems to be a fairly well established rule that it takes fully 50 years to complete an engineering meteorological cycle. This was illustrated, for example, at Topeka, Kans. When the Melan Bridge was built over the Kansas River in 1897, the City Engineer's records for 25 years showed a maximum rise of 15 ft. from L. W. to H. W., and the oldest inhabitant, who claimed that he had seen the river go clear out of its banks and flood the entire valley prior to 1850, was deemed to be in his dotage and untrustworthy. In 1903, however, the recorded variation of 15 ft. between low and high water was changed to 28 ft.; asphalt pavements were 6 ft. under water; the oldest inhabitant was vindicated, and the river proved what every one should have known—that the high-water mark, in an alluvial valley which has been built up by sediment dropped by the floods, is necessarily higher than the ground surface of the valley—regardless of the records of the City Engineer's office."

While these remarks bear out, in a general way, the author's conclusions, the writer acknowledges that his 50-year "meteorological cycle" is subject to amendment.

Tables 1 and 2 give, at a glance, an idea of what may be reasonably expected, and, from this time forward, the Profession will be able to act more intelligently in the design of waterways and storage reservoirs. The writer has been collecting data in regard to maximum rainfall for 30 years, and the two subjects together are an interesting study.

There are plenty of bridges and spillways which may stand up under 1.80 times the annual flood, but will go out when 3.40 times the annual flood comes.

The writer, having kept a record of intensity of rainfall in the United States for the past forty years, offers a table and a diagram (Fig. 7) of the same, and begs the author's pardon for injecting "Rainfall" into "Flood Flows," and for applying his table backward for constructing three of the curves on the diagram.

TABLE 31.

By Author's Table 2.		Rainfall intensity ratios.
Years.	Flood ratios.	
50	2.36	1.00
10	1.80	0.76
5	1.56	0.66
1	1.00	0.42

Mr. Hinckley.

The maximum rates of rainfall from which the 50-year curve was constructed several years ago (without any attempt to evolve an equation for it), are as follows: 14 min., 16.90 in. per hour; 3 hours, 5.33 in. per hour; 11 hours, 2.70 in. per hour.

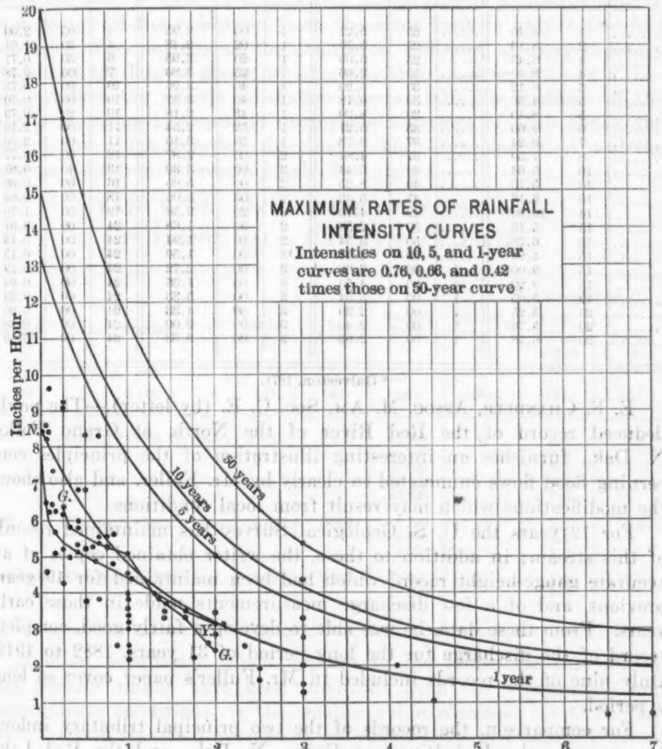


FIG. 7.

It is interesting to note how closely the resulting 1-year curve fits two downpours which have been reported, from New York for October 16th, 1913, and Galveston, for October 22d, 1913, which are designated on Fig. 13 as "N-Y." and "G-G.", respectively.

The author has given us a valuable paper which has propounded to every hydraulic engineer the question: "For how many years will your works stand?"

Mr. Hinckley.

TABLE 32.—A FEW SAMPLES OF MAXIMUM RATE OF RAINFALL, IN INCHES PER HOUR.

Hr.	Min.	Inches per hour.	Hr.	Min.	Inches per hour.	Hr.	Min.	Inches per hour.	Hr.	Min.	Inches per hour.
....	1	8.40	......	25	5.21	1	00	2.95	4	05	2.00
....	2	8.10	......	25	5.76	1	02	2.48	4	30	1.01
....	4	6.45	......	25	6.00	1	20	3.95	6	30	0.71
....	4	8.40	......	30	3.66	1	25	3.80	7	00	0.78
....	5	6.12	......	30	5.20	1	40	3.26	10	00	0.75
....	5	4.56	......	30	6.78	1	40	3.37	10	00	0.70
....	5	8.40	......	30	8.20	1	43	2.18	10	20	0.73
....	6	9.60	......	35	5.21	1	52	2.54	11	00	2.10
....	7	6.24	......	37	3.78	1	59	3.10	11	00	2.70
....	8	7.50	......	37	4.84	2	00	2.87	12	00	0.77
....	10	5.64	......	40	5.44	2	00	3.30	13	40	0.81
....	10	5.10	......	40	8.25	2	00	5.00	16	00	0.86
....	10	6.18	......	45	5.43	2	03	3.06	18	00	0.82
....	14	16.90*	......	49	4.75	2	20	2.58	19	50	1.10
....	15	5.16	......	50	2.58	2	30	1.68	24	00	0.57
....	15	6.28	......	50	5.54	2	30	1.94	24	00	0.13
....	15	4.24	......	59	4.44	3	00	1.50	24	00	0.15
....	15	9.00	1	00	3.68	3	00	2.72	24	00	0.19
....	15	9.20	1	00	4.50	3	00	1.36	24	00	0.24
....	19	5.05	1	00	5.04	3	00	5.33	24	00	0.23
....	20	5.25	1	00	2.30	3	00	4.33	24	00	0.26
....	20	5.76	1	00	2.40	3	00	2.00	24	00	0.48
....	20	6.78	1	00	2.96	3	00	3.33	24	00	0.40

* Galveston, 1871.

Mr. Chandler.

E. F. CHANDLER, ASSOC. M. AM. SOC. C. E. (by letter).—The newly deduced record of the Red River of the North, at Grand Forks, N. Dak., furnishes an interesting illustration of the principles concerning flood flows enunciated so clearly by Mr. Fuller, and also shows the modifications which may result from local conditions.

For 12 years the U. S. Geological Survey has maintained records of this stream; in addition to these, the writer obtained copies of an accurate gauge-height record which had been maintained for 19 years previous, and of a few discharge measurements made in those early years. From these data, he was able to develop a fairly good, complete record of the discharge for the long period of 31 years, 1882 to 1912. Only nine of the records included in Mr. Fuller's paper cover so long a period.

For comparison, the records of the two principal tributary inflowing streams, the Red River, at Fargo, N. Dak., and the Red Lake River, at Crookston, Minn. (which have been observed for 11 and 10 years, respectively), may be considered at the same time. The complete records to date for these stations are shown in Table 33, and might be included in Table 18.

The largest floods did not occur in the same years at these stations, notwithstanding the fact that the latter two are tributary to the first; therefore, all three records may be considered.

In each case, the largest flood was less than that which would be given by the logarithmic formula, on the basis of the median floods.

It seems that this might reasonably have been expected from the topographic characteristics of the Red River Valley, the river and all its main tributaries flowing through remarkably level prairies, usually in channels only a few hundred feet wide and from 10 to 40 ft. below the prairie level. On each side the prairie is almost as smooth as a floor, and the rise away from the river bank is only a few feet per mile. The capacity of the channel is sufficient to carry ordinary floods, but the largest floods, which occur only once or twice in a decade, overflow the prairie in shallow sheets; thus the regimen of the stream is changed essentially during the extremes of such floods, the rate of flow much retarded, the time of high stage increased, and the extreme maximum correspondingly diminished. Mr. Chandler.

TABLE 33.—VALUES FOR COEFFICIENT, C, IN FORMULA $Q = CA^{0.8}$ (1 + 0.8 LOG. T).

Stream. (1)	Station. (2)	Area, A. (3)	Q (Ave.). (4)	Larger flood, Q. (5)	Period, years, T. (6)	Values of C. From larger Q. (7)	Values of C. From Q (Ave.). (8)
Red.......	Grand Forks, N. Dak..	25 000	17 440	42 400 40 800 37 500 33 400 32 920	31	5.8 6.5 6.7 6.8 7.0	5.3
Red.......	Fargo, N. Dak.........	6 020	2 809	6 090 5 800 4 250	11	3.1 3.5 3.5	2.6
Red Lake.	Crookston, Minn......	5 320	7 478	14 200 13 600 10 300	10	8.3 9.3 9.8	7.8

Another peculiarity which appears in the tables for this region, and perhaps might have been predicted from the topographic and meteorologic conditions, is the smaller value obtained for C in Column 8 (from the flood average of all years) than in Column 7 (from the largest floods). The Red River Valley is near the "semi-arid" region, and the river separates a prairie State from one largely forested. At one margin of the drainage area, the mean annual rainfall is about 16 in., at the other margin 24 in.; the long-period mean annual run-off is about 1 in. at one side and 4 in. at the other. Therefore, in a region where evaporation consumes almost the whole of the precipitation, and the run-off is so small a remainder, after any season of deficient rainfall, the run-off becomes much less even than the normal, and then remains comparatively steady until the ground storage is replenished by surplus rainfall, and at such times it is little affected

Mr. Chandler.

by ordinary rainstorms. Thus, though the greatest recorded flow at each of these stations is ten or twelve times the long-period mean run-off, there are many years when there is no noticeable flood, and three or four years out of every ten when the largest flood of the year is less than three times the mean discharge. As a result, Q (Ave.), the average of the annual floods, gives a smaller coefficient than that obtained from the larger floods individually.

It is probably the same cause, namely, the small maximum discharge of some years, which gives rise to a different logarithmic factor (in a formula deduced only from the streams of this region) than that deduced by Mr. Fuller from the average of streams scattered through the whole country. Instead of the factor, 0.80 log. T, in the formula, $Q = C\,A^{0.8}\,(1 + 0.80 \log. T)$, the largest flood at each of these three stations would give 0.95 log. T, 1.12 log. T, and 0.90 log. T. In this region, as mentioned previously, the greatest floods do not seem to be as great in comparison with those of secondary magnitude as is normal in other regions; if, therefore, we omit a few of the largest from consideration, the following floods would give factors from 1.10 log. T to 1.30 log. T.

Mr. Fuller states that the rivers of the Hudson Bay drainage varied so widely among themselves that no logarithmic plottings of them were published. The Red River Valley portion of the basin, however, including these three records and numerous other shorter or less accurate records, shows fair uniformity. As Mr. Fuller explains, in some cases local conditions may be expected to modify the application of the formula or to change it to some extent, and such seems to be the case here. The importance of this case would not be great enough to justify this addition to the discussion, were it not for the length of the record—31 years. We see illustrated how useless or erroneous it would be to attempt to deduce figures concerning the flow of an unknown stream where no actual measurements had been made in the region; but, on the other hand, by using the records as a basis for small necessary modifications, the principles enunciated by Mr. Fuller seem, in general, to apply in this region also, and to furnish excellent starting points for more detailed studies.

Mr. Hazen.

ALLEN HAZEN, M. AM. SOC. C. E. (by letter).—This is a most important paper, because, as far as the writer knows, it is the first attempt to apply the principles of probabilities to the flood problem.

The writer has followed the author's work in detail, and believes that his methods are sound. As time goes on, data covering longer periods and more streams may change some of the numerical values; but the underlying idea of treating the recurrence of floods as a matter of probabilities, to be determined by an examination of the records of many streams, will stand.

Mr. Hazen.

When the writer first took up the study of this problem, several years ago, he believed that it would be safer to use as a basis the records of a few streams which covered long periods, where knowledge of flood flows had been obtained from records of the height of water over the crests of substantial dams above the influence of back-water, and where there could be no question as to the accuracy of the results. On the basis of a limited quantity of selected data, tentative values were reached. The author afterward extended the investigation to include short-term records of many more streams; but the numerical values resulting from this broader study were fully in accord with those first obtained by the writer from the small quantity of selected data.

Taking into account the many elements of error which exist or can be imagined to exist in stream gaugings, and especially the fact that estimates for extreme floods in many or most cases are based on extrapolations from rating curves, often at some distance above the highest stage for which gaugings have actually been made, the writer would feel disposed, in case they differed, to attach quite as much importance to the results obtained from his first study. As the more extended studies presented by the author are entirely in accord with the writer's earlier ones, this question does not require discussion, and the writer feels confidence in the general accuracy of the results.

In another paper to be presented to the Society, the writer has described a graphical method of representing data of the same general character as those used by the author, based on the probability curve, or, as it is otherwise called, the normal law of error. A new kind of cross-section paper is made, in which the spacing of the lines in one direction is computed from tables of the probability curve, so that figures representing the summation of that curve plotted on it fall in a straight line.

From a study of the author's data, it is clear that the relation between flood flows and the normal law of error is such that this probability paper could have been used for his study, had it been available. Its use might have facilitated some of the work, but there is no reason to think that it would have changed to a significant extent any of his conclusions. In fact, the paper the author has used for his study corresponds so closely with that part of the probability paper in which most of his data would fall, that the differences would not have been important in the graphical method.

The figures for maximum flood flows of the same stream through a series of years clearly bear a relation to the normal law of error, but they do not follow it exactly. When plotted on probability paper they form what is called a skew curve. The variations downward are more numerous than the variations upward, but are smaller in magnitude.

Mr. Hazen. It is interesting to note that if the logarithms of the numbers representing the several floods are used instead of the numbers themselves, the agreement with the normal law of error is closer. In other words, the variations up and down from the mean follow a geometric rather than an arithmetical ratio.

If the records of the floods of a number of streams were available for very long terms, it would be possible to find a "standard variation" and a "coefficient of variation" from the records of each stream, which would be an index of the degree of variability to be expected in its floods, and indirectly of the size of the floods to be anticipated. There is no reason to suppose that the coefficients of variation in flood flows for different streams would be the same. However, short-term data are not sufficient to determine the coefficient of variation with the requisite degree of accuracy to make this comparison; and the assumption tacitly made by Mr. Fuller, that the coefficient of variation of all the streams in one geographical group will be the same, may be found to be nearly in accordance with the facts. At any rate, the data now available are not sufficient to justify a further classification.

The constants found in the proposed flood-flow formula deduced for rivers in different parts of the United States throw some light on this point. If the coefficients of variation in flood flows in different parts of the country differed considerably, there would be differences in these constants. The smallness of the differences found by the author indicates a surprising degree of uniformity in flood conditions for a large part of the country, and this uniformity by districts may be taken as an indication of the probable absence of wide variations among different streams in the same district.

To attempt to secure actual coefficients from the data at hand, and to compare with one another streams having records no longer than are now available, is like attempting to find the probability of a house burning down from the records of single houses.

In the practical application of the author's formula, the value of the coefficient to be multiplied by $A^{0.8}$ to produce the average annual flood, is a matter of the first importance. In only a limited number of cases will it be possible to base such estimates on actual gaugings of the stream for which estimate is to be made. In a majority of cases, it will be necessary to depend on the records of other streams more or less similarly situated.

On Fig. 8, an outline map of the United States, the coefficients for the streams, for which Mr. Fuller presents data, have been plotted. This map serves conveniently to show what data are available at this time. It is seen that there are large parts of the United States which are not covered at all, and others which are covered most inadequately. In forming judgment as to the probable value of the

Mr. Hazen.

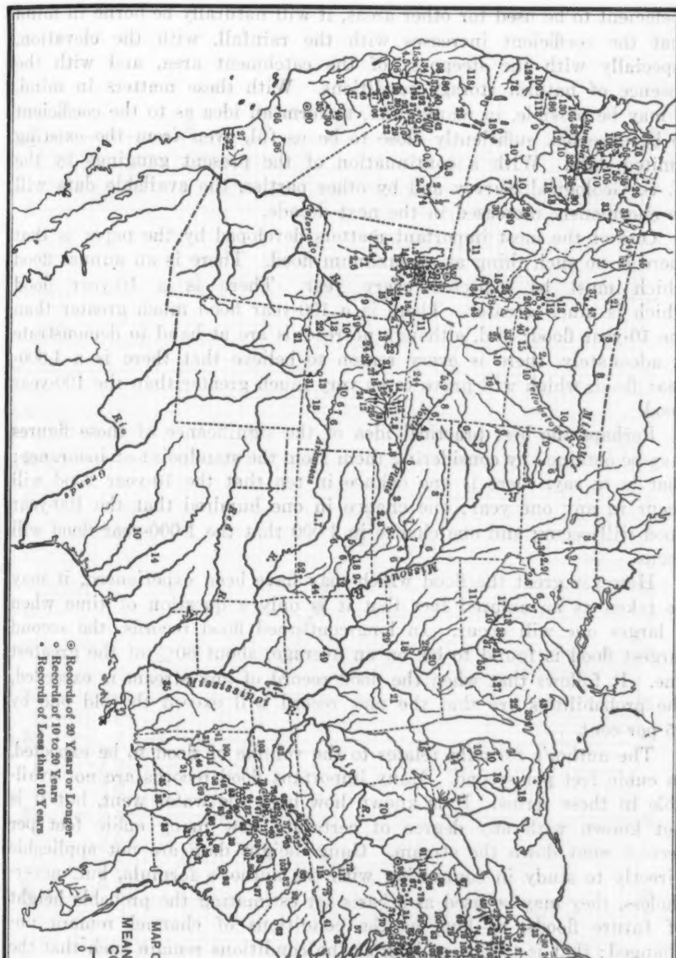


FIG. 8.

Mr. Hazen. coefficient to be used for other areas, it will naturally be borne in mind that the coefficient increases with the rainfall, with the elevation, especially with the steepness of the catchment area, and with the absence of natural storage conditions. With these matters in mind, it may be possible, in many cases, to form an idea as to the coefficient to be expected, sufficiently close to be useful, even from the existing limited data. With a continuation of the present gaugings by the U. S. Geological Survey and by other parties, the available data will be enormously increased in the next decade.

One of the most important matters developed by the paper is that there is no such thing as a maximum flood. There is an annual flood which must be expected every year. There is a 10-year flood which is much greater. There is a 100-year flood much greater than the 10-year flood; and, although no records are at hand to demonstrate it adequately, there is every reason to believe that there is a 1 000-year flood, which will prove to be very much greater than the 100-year flood.

Perhaps the best practical idea of the significance of these figures may be obtained by considering them from the standpoint of insurance; that is to say, there is one chance in ten that the 10-year flood will occur in any one year; one chance in one hundred that the 100-year flood will occur; and one chance in 1 000 that the 1 000-year flood will occur.

However great the flood which may have been experienced, it may be taken as an assured fact that it is only a question of time when a larger one will occur. In long-continued flood records, the second largest flood is found to be, on an average, about 80% of the greatest one. It follows that when the flood record of any stream is exceeded, the probabilities are that the new record will exceed the old one by 25 per cent.

The author's formula relates to the volume of flood to be expected, in cubic feet per second. Many important flood records are not available in these terms. It is known how high the water went, but it is not known with any degree of certainty how many cubic feet per second went down the stream. Gauge-height data are not applicable directly to study in connection with the author's formula, but, nevertheless, they may be used as a basis for estimating the probable height of future floods, as long as the conditions of channel remain unchanged; that is to say, as long as the conditions remain such that the same volume of discharge may be expected to produce the same or nearly the same height of water. Within this limit, as the quantity of discharge, or some function of it, follows approximately the normal law of error, and as the height of water is a function of the quantity of discharge, it follows that the height of water, or some function of it, will follow approximately the normal law of error; and, with constant

Mr. Hazen.

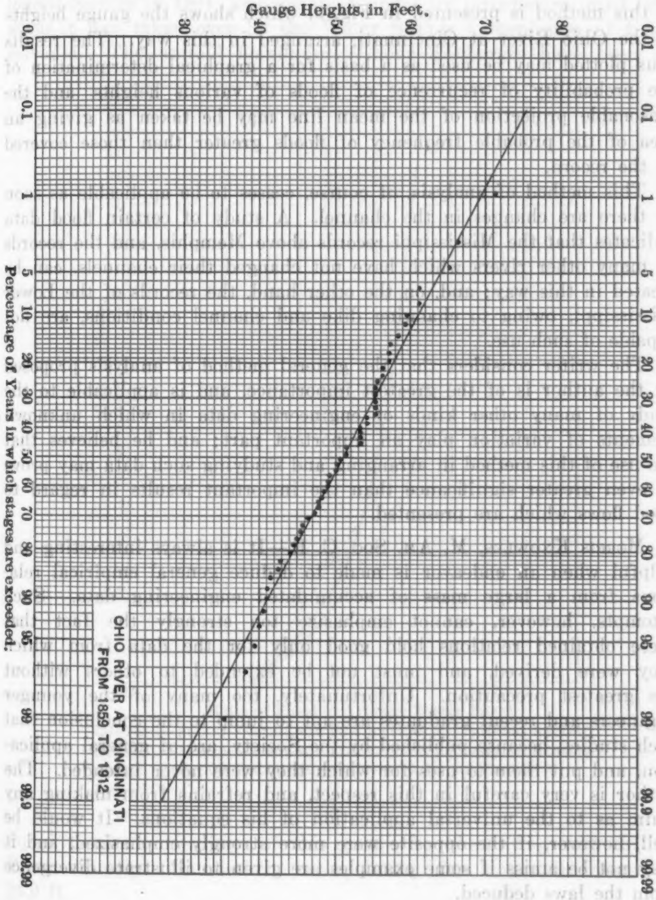


FIG. 9.

Mr. Hazen. channel conditions, the records of gauge heights for a term of years may be arranged in the order of magnitude and plotted, as the author has plotted his flood data, on logarithmic paper, or as the writer would prefer, on the probability paper previously described. An illustration of this method is presented in Fig. 9, which shows the gauge heights of the Ohio River at Cincinnati, arranged in this way. The results thus plotted may be used as a basis for a graphical determination of the probability of recurrence of floods of various heights, and the reasonable projection of the mean line may be taken as giving an idea of the probable frequency of floods greater than those covered by the record.

This method of analysis, of course, ceases to be applicable as soon as there are changes in the channel. A study of certain flood data indicates that the Mississippi records above Memphis, and the records of many other rivers which have not changed their channels, can be treated in this way; and, on the other hand, the records of the Lower Mississippi, owing to changing dike and channel conditions, are not capable of such use.

The writer considers that the general method of analysis proposed by the author is of the greatest importance, and is applicable to the study of many other kinds of engineering data in which unknown elements of variation play an important part; and he believes that the use of this method in arranging and studying such data may prove of even greater significance than the important results in regard to flood flows which are presented.

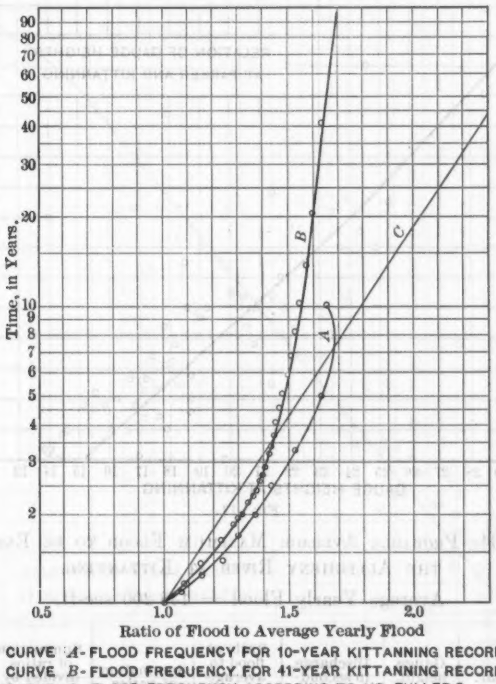
Mr. Knowles. Morris Knowles, M. Am. Soc. C. E.—It is always interesting and helpful when an endeavor is made to deduce general empirical relations from a large mass of accumulated engineering data. Such attempts, however, cannot emphasize too strongly the fact that these obtained relations hold good only for the data from which they were derived, and must not be extended to others without the greatest precaution. Unfortunately, too many of the younger engineers and recent graduates are apt to jump to the conclusion that such studies, because published by the Society, are of general application, and put them to uses for which they were never intended. The author is very careful in this respect, and refrains from making any claim as to the universal application of his equations. It would be well, however, if the opposite were more strongly emphasized, and it may not be amiss if some examples are given to illustrate divergence from the laws deduced.

Point is given to these remarks by the fact that, at about the time this paper was published, the speaker was interested in investigating the magnitude and frequency of floods on the Allegheny River at Kittanning, Pa., and was led to check up the relation between average

Mr. Knowles.

and maximum floods as shown by the records with that indicated by the author. The results may be of some interest.

Gaugings of the Allegheny River at Kittanning are now available for 10 years. The gauge was established by the United States Geological Survey in 1904, and remained under its jurisdiction until 1907, when it was taken over by the Water Supply Commission of Pennsylvania. The annual maximum 24-hour discharges have been de-



CURVE *A*- FLOOD FREQUENCY FOR 10-YEAR KITTANNING RECORD
CURVE *B*- FLOOD FREQUENCY FOR 41-YEAR KITTANNING RECORD
CURVE *C*- FLOOD FREQUENCY ACCORDING TO MR. FULLER'S EQUATION.

FIG. 10.

duced from the readings of this gauge, by the rating table of the Water Supply Commission, extrapolating for stages in excess of 25.0 ft.

Following the methods used by the author in compiling Table 5, the results shown in Table 34 are obtained.

Plotting the values in Columns 7 and 8 on logarithmic paper, as was done for Curve *A* on Fig. 1, gives the curve, *A*, on Fig. 10. This

Mr. Knowles.

is interesting in that it is not a straight line, as is the case for all the rivers for which plots were made by the author.

With the idea of studying the effect of increasing the period of observation, an attempt was made to complete the Kittanning record for a total of 41 years, by studying the relation of recorded gauge readings at Kittanning with those at Freeport and Parker. The gauge

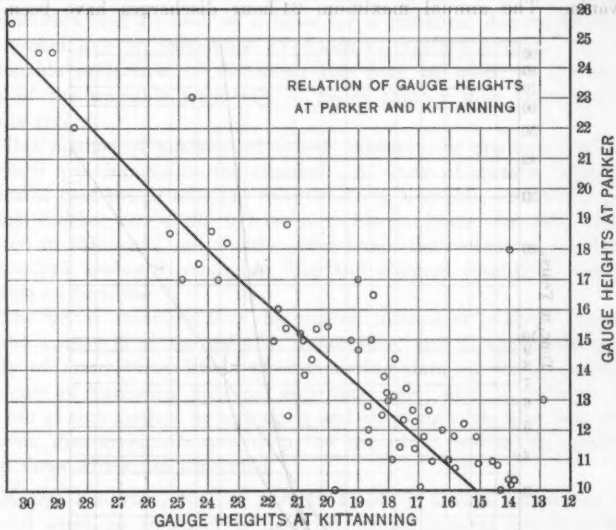


FIG. 11.

TABLE 34—PROBABLE AVERAGE MAXIMUM FLOOD TO BE EXPECTED IN THE ALLEGHENY RIVER AT KITTANNING.

Average Yearly Flood = 148 200 sec-ft.

No. of flood, in order of magnitude. (1)	Year. (2)	Gauge height, in feet. (3)	Discharge, in second-feet. (4)	Ratio of flood to average yearly flood. (5)	Summation of ratios. (6)	Summation of ratios divided by number of flood. (7)	Time, in years. (8)
1	1913	29.5	247 500	1.67	1.67	1.67	10
2	1905	28.2	234 200	1.58	3.25	1.63	5
3	1908	24.3	190 200	1.30	4.55	1.52	3.3
4	1912	23.0	172 000	1.16	5.71	1.43	2.5
5	1909	22.8	168 800	1.14	6.85	1.37	2.0
6	1910	20.8	139 100	0.94	7.79	1.30	1.7
7	1911	19.1	118 000	0.79	8.58	1.23	1.4
8	1907	15.9	86 500	0.59	9.17	1.15	1.25
9	1906	14.0	69 100	0.47	9.64	1.07	1.1
10	1904	12.4	56 100	0.38	10.02	1.00	1.0

at Freeport, 26 miles below Kittanning, was established in 1873, and that at Parker, 39 miles above Kittanning, in 1884. As Freeport is at the mouth of the Kiskiminetas River, one of the largest tributaries of the Allegheny, and as no important tributaries enter at Parker or between Parker and Kittanning, it was thought that a more constant relation might be expected between the gauges at Kittanning and Parker, than between those at Kittanning and Freeport. Mr. Knowles.

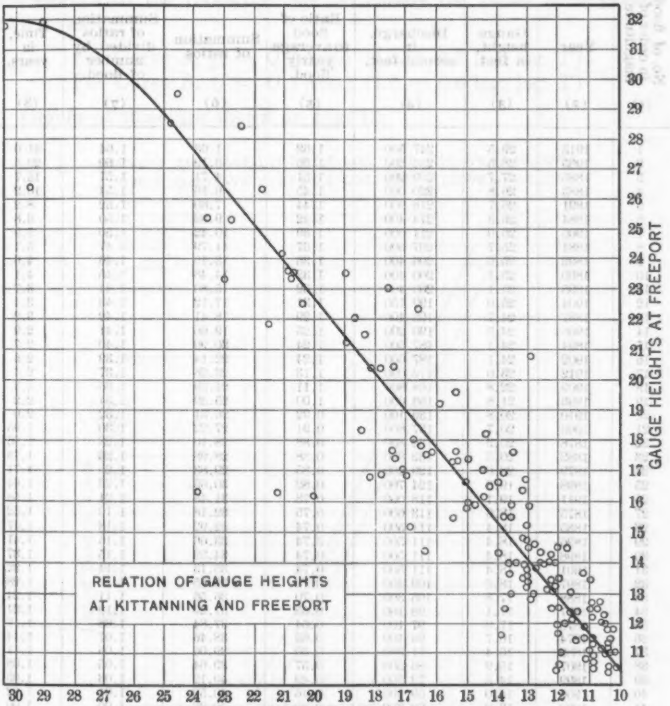


FIG. 12.

All gauge readings of 10 ft. or more at Kittanning since 1904, therefore, were plotted, with the corresponding gauge readings at Parker and Freeport, as shown in Figs. 11 and 12. Estimated Kittanning gauge heights were taken from the Freeport curve for the period, 1873-84, and from the Parker curve for 1884-1904. In plotting the Freeport-Kittanning curve, all points representing local floods in the Kiskiminetas River were given less weight.

Mr. Knowles.

Estimating the corresponding discharges from the same rating curve, and proceeding as before, gives the results shown in Table 35.

TABLE 35.—PROBABLE AVERAGE MAXIMUM FLOOD TO BE EXPECTED IN THE ALLEGHENY RIVER AT KITTANNING, 1873-1913, INCLUSIVE.

Average Yearly Flood = 151 300 sec-ft.

No. of flood in order of magnitude. (1)	Year. (2)	Gauge height, in feet. (3)	Discharge, in second-feet. (4)	Ratio of flood to average yearly flood. (5)	Summation of ratios. (6)	Summation of ratios divided by number of flood. (7)	Time, in years. (8)
1	1913	29.5	247 500	1.63	1.63	1.64	41.0
2	1905	28.5	234 200	1.56	3.19	1.60	20.5
3	1886	27.7	229 200	1.51	4.71	1.57	13.7
4	1883	26.8	220 000	1.45	6.16	1.54	10.2
5	1891	26.7	218 900	1.44	7.60	1.52	8.2
6	1884	26.3	214 400	1.42	9.02	1.50	6.8
7	1900	26.0	211 000	1.39	10.42	1.50	5.9
8	1881	25.7	207 500	1.37	11.79	1.47	5.1
9	1892	25.6	206 400	1.36	13.15	1.46	4.6
10	1890	25.1	200 400	1.32	14.48	1.45	4.1
11	1895	25.1	200 400	1.32	15.80	1.44	3.7
12	1904	25.0	199 150	1.31	17.12	1.43	3.4
13	1887	24.7	195 400	1.29	18.41	1.42	3.2
14	1908	24.3	190 200	1.25	19.66	1.41	2.9
15	1894	24.1	187 600	1.24	20.90	1.40	2.7
16	1902	24.1	187 600	1.24	22.14	1.39	2.6
17	1912	23.0	172 000	1.13	23.28	1.37	2.4
18	1909	22.8	168 800	1.11	24.39	1.36	2.3
19	1893	21.8	153 000	1.00	25.39	1.34	2.2
20	1910	20.8	139 100	0.92	26.31	1.32	2.0
21	1903	20.7	137 800	0.91	27.22	1.30	1.95
22	1878	20.3	132 800	0.88	28.10	1.28	1.86
23	1882	20.3	132 800	0.88	28.98	1.26	1.78
24	1879	20.0	129 200	0.85	29.83	1.25	1.71
25	1898	19.6	124 700	0.82	30.65	1.23	1.64
26	1911	19.1	118 000	0.78	31.43	1.21	1.58
27	1875	18.6	113 600	0.75	32.18	1.19	1.52
28	1885	18.4	111 500	0.74	32.92	1.18	1.47
29	1896	18.4	111 500	0.74	33.66	1.16	1.41
30	1898	18.4	111 500	0.74	34.39	1.15	1.37
31	1901	18.4	111 500	0.74	35.13	1.14	1.32
32	1897	18.2	109 400	0.72	35.85	1.12	1.28
33	1888	17.8	105 300	0.70	36.56	1.11	1.24
34	1877	17.1	98 100	0.65	37.20	1.10	1.21
35	1876	17.0	97 100	0.64	37.84	1.08	1.17
36	1874	16.7	94 100	0.62	38.46	1.07	1.14
37	1889	16.4	91 300	0.60	39.06	1.06	1.11
38	1907	15.9	86 500	0.57	39.64	1.05	1.08
39	1899	14.5	73 500	0.49	40.12	1.03	1.05
40	1906	14.0	69 100	0.46	40.58	1.01	1.02
41	1880	13.0	60 800	0.40	40.98	1.00	1.00
			6 202 850	Average = 151 300			

Plotting Columns 7 and 8 of Table 35 on logarithmic paper gives Curve B on Fig. 10. This, also, is not a straight line, and though its upper portion is nearly straight, the line does not pass through the point ($R = 1$, $T = 1$) as would be required by the equation:

$$R = 1 + 0.8 \log. T.$$

Curve *C*, Fig. 10, shows the author's equation, and indicates that its use would yield results too large, or on the side of safety, as compared with the computed expectancies, for all periods in excess of 5 or 6 years. Mr. Knowles.

It may not be without interest to add that several attempts were made to obtain an equation which would fit Curves *A* and *B*, and it was found that the following checked closely for values of T greater than 2 years:

$$\text{For Curve } A,\ Q = Q\ (\text{Ave.})\ (1.7 + 0.6 \text{ log. log. } T)$$
$$\text{For Curve } B,\ Q = Q\ (\text{Ave.})\ (1.6 + 0.6 \text{ log. log. } T)$$

Graphs of these are shown on Fig. 13.

CURVE *A*. $Q=Q$(AVE.) (1.7+0.6 LOG. LOG. T)10-YEAR RECORD.
CURVE *B*. $Q=Q$(AVE.) (1.6+0.6 LOG. LOG. T)41-YEAR RECORD.

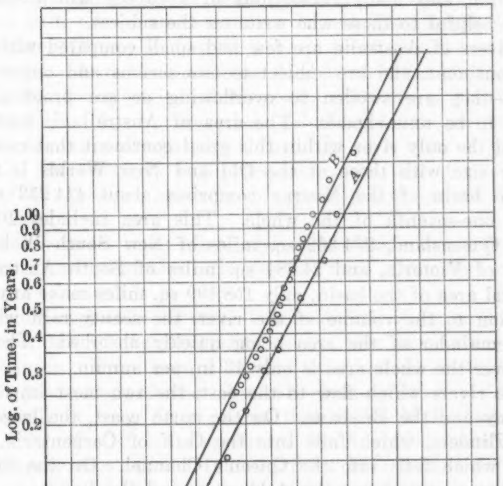


FIG. 13.

It should be noted in passing that the average annual flood computed by the speaker for the 10-year record at Kittanning, or for any conservative 7 years of that record, is substantially greater than that given by the author for the Allegheny River at Kittanning in Table 16. No doubt this is due to the use of a different rating curve in connection with the gaugings.

Mr. Bellamy.

HERBERT E. BELLAMY, ASSOC. M. AM. SOC. C. E. (by letter).—This paper is most interesting and well worthy of a place in the *Transactions* of the Society. The problems associated with the great rivers of America, and the great forces of Nature brought into operation by the enormous rainfalls of that country, spread over great catchment areas, are subjects of much interest to engineers. The writer is sure that those resident in Australia especially will be very thankful to the author for his valuable paper, and also for the lucid manner in which he has presented it. From a careful perusal, it would seem, to those studying the question of the discharge of rivers in flood times, that the tabulated statements submitted by Mr. Fuller, comparing one river with another, supply information which has long been needed. This comparison is of great value. It suggests a method which is simple, and at the same time possesses certain elements of mathematical precision and many indications of accuracy which will be exceedingly helpful to those who write on the subject.

The rivers of Australia are few and small compared with the size of the Continent, and are subject to two serious and opposite disadvantages—they are swollen to overflowing or are practically dried up so as to be unnavigable. The area of Australia is 2 950 000 sq. miles, and the only river within this great continent that can be compared for size with those of the Old and New Worlds is the Murray. The basin of the Murray comprises about 414 253 sq. miles, or about one-seventh of the whole. This area includes 104 525 sq. miles of Queensland, 234 362 sq. miles of New South Wales, 50 979 sq. miles of Victoria, and 24 384 sq. miles of South Australia; but, of the total area of the basin, only 158 499 sq. miles make any effective contribution to the volume of the river, the scanty rain which falls on the remainder of the area being quickly absorbed. The average rainfall over the whole area is only 13 in. per annum.

Of the rivers which flow to the east, the two most important are the Fitzroy and the Brisbane. On the north coast, the largest rivers are the Flinders, which falls into the Gulf of Carpentaria, and the Victoria, which falls into the Queen's Channel. On the west coast, the best-known rivers are the Ashburton and the Swan.

The misfortune of Australia, as regards rainfall, is that the mountain ranges, which act as condensers, lie so near the east coast. The result is that the narrow coastal plain gets more rain than it needs, and, when the rain-bearing winds from the Pacific have crossed the mountains and table-lands into the interior, the great heat there dissipates the clouds and does not permit them to condense into rain.

In attempting to form a rule for flood discharges for Australian rivers, it will be found that there are as many exceptions as there are rivers, and further that the flood discharge in each river varies according to the precise locality in which it is measured. The writer

considers that the question of the fresh-water floods of Australian rivers in relation to the areas and physical features of their basins is one of those multiform problems which can only be solved by special attention to the peculiar circumstances of each particular case. One of the chief wants experienced by the water engineer in Australia arises from the scantiness of reliable data regarding the occasional floods to which the rivers especially, and parts of the country generally, are subject. There is probably much valuable information in the hands of a few engineers now engaged on public works throughout the country, but, except in one or two cases, it has never been collected for reference. In any case there is great difficulty in securing reliable evidence of the levels attained by great floods which occurred more than 40 or 50 years ago. This difficulty has been found by the writer on several occasions when desiring to fix permanent levels for new pumping stations to be constructed on the banks of rivers in connection with town water supplies. In one case, for Rockhampton, on the Fitzroy River, he deemed it advisable to fix the engine-house floor level 3 ft. above the maximum flood level, although flood records were available for a period of 47 years. Records of the heights of various floods in this river have been tabulated by the writer.* Mr. Bellamy.

On the eastern coast the flood discharge of a river is greater per square mile for relatively small drainage areas than for larger ones, because of the greater intensity of precipitation on the former in time of storm.

The author states that: "In studying the data, a few rivers, located principally in arid and semi-arid regions, were eliminated on account of unusual conditions."

It is more especially to these latter conditions, in so far as they pertain to a few of the Australian rivers, that the writer wishes to direct attention. From even a cursory examination of Table 36, and the brief description of the physical characteristics of Australian rivers, it will at once be apparent that it would be impossible to establish a set of coefficients applicable for Australian conditions.

The Murray rises, as the Indi, in Pilot Mountain, 5 000 ft. above sea level, and, receiving a large number of tributary mountain torrents fed by the snows of the Muniong and Bogong Ranges, flows swiftly down from the table-land on the lower plain, falling 4 500 ft. in its first 300 miles. Toward Albury it is joined by the Mitta and Kiewa, and, between Albury and Wentworth, the affluents of the Murray are the Ovens, Goulburn, Campaspe, and Loddon from the south, and the Murrumbidgee and Darling from the north. The fall of the river from Albury downward varies from 9 to 4 in. per mile. From Wentworth, however, to Lake Alexandrina, the fall is only 3 in. per mile. Lake Alexandrina has an area of 288 sq. miles, and the outlet

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXIII.

Mr. Bellamy. for the Murray is through the Goolwa and Coorong Channels, which unite at Mundoo Island, and form one channel to the sea. The mouth of the Murray resembles many of the Australian bar-bound coastal rivers, and proposals have been made from time to time to make it navigable; but, on account of the formidable and costly difficulties to be encountered, nothing has yet been done.

The Goulburn River is the largest and most important Victorian tributary of the Murray. This river, which flows into the Murray at a point 880 miles from Morgan, and 676 miles from the South Australian boundary, for the twelve years ending with 1903, inclusive, had a maximum discharge of 37%, a minimum of 16%, and a mean of 23%, of the discharge at Morgan. The mean monthly discharge was from 30 to 70% in 1884, and from 21 to 62% in 1887, of that of the Murray at Echuca, which is 10 miles below the junction. The description of the Upper Murray is, in many respects, applicable to this river. It takes its rise in the Dividing Range, near Wood's Point, where the summits reach an elevation of 5 000 ft. The drainage area is about 9 000 sq. miles, about 1 500 sq. miles being in mountainous country of considerable elevation. This portion of the catchment area is rocky and precipitous, and a large proportion of the rainfall is discharged. The winter volume of the river is large, and the melting snows maintain the discharge far into the summer. About 5 200 sq. miles of the total catchment are effective, the remainder being non-contributing. The term, Goulburn Valley, is limited, by popular usage, to the plain which extends from the Town of Murchison northward to the Murray.

The Lachlan River possesses a reputation for irregularity of flow which is, perhaps, not paralleled by any other river in Australia. As illustrative of the great fluctuations which take place in the volume discharged by the river, the records for 1900 and 1902 might be quoted. In July, 1900, a sudden downpour of rain, aided by melting snows in the ranges forming the upper portion of the Lachlan gathering ground, caused the river to rise from 19 ft. to 46 ft. 8 in. in 26 hours at Cowra. The estimated discharge at this height was 1 800 000 cu. ft. per min. This volume was maintained for nearly two days, after which the river gradually subsided to its normal level. The total discharge for July, 1900, exceeded 19 000 000 000 cu. ft. The flood of 1894, although not at any time reaching the maximum recorded in July, 1900, was more sustained, as the river was in flood for four months of the year, and in that period not less than 66 600 000 000 cu. ft. passed the gauging station. By way of comparison, it may be mentioned that this was 60% more than the volume discharged by the Murray at Albury for the whole year 1902. During the drought year, 1902, the Lachlan was practically a chain of waterholes, and

Mr. Bellamy.

TABLE 36.

Name of river.	Situation of gauging station.	Maximum Annual Volume Passing Gauging Station.		Minimum Annual Volume Passing Gauging Station.		Maximum Recorded Discharge per Minute.		Minimum Recorded Discharge per Minute.		Catchment area, in square miles.	General remarks.
		Cubic feet.	Year.	Cubic feet.	Year.	Cubic feet.	Year.	Cubic feet.	Year.		
Murray........	Albury	264 383 000 000	1894	41 091 000 000	1902	3 214 000	1880	17 500	Feb. 1903		
Murray........	Mildura......	1 141 728 000 000	1870	57 517 000 000	1902	6 000 000	1870	14 400	Apr. 1903		
Murray........	Morgan......	1 067 000 000 000	1890	105 000 000 000	1902	4 000 000	1890	44 500	Apr. 1903	......	There are no means of estimating the proportion of rainfall discharged by the Murrumbidgee, as the records do not include the highest peaks of the Dividing Range, where the heaviest falls of rain and snow occur. Gaugings from 1885.
Murrumbidgee	Hay.........	400 000 000 000	1894	18 500 000 000	1902	762 000	1894	12 000	Feb. 1903		
Lachlan	Forbes	92 600 000 000	1894	1 024 000 000	1902	1 800 000	1870	2 000 (mean only)	1902	20 159	Only in times of high flood that the waters of the Lachlan reach the Murrumbidgee. Ceased to flow 9 months in 1902.
Darling........	Wilcannia...	717 000 000 000	1890	700 000 000	1890	....		....	......	......	From February, 1902, to January, 1903, the river ceased to flow on account of the drought. Gaugings from 1885.
Goulburn......	Murchison...	168 000 000 000	1887	27 753 000 000	1902	1 923 330	1887	4 000	Mch. 1902	9 000	During 1887 it is estimated that 35% of the total rainfall on the catchment area was discharged.
Ovens..........	Waugaratta.	82 948 000 000	1894	6 118 000 000	1902	658 000	1894	....	......	3 000	During 1902 it is estimated that only 5% of the total rainfall on the catchment area was discharged.
Mitta Mitta....	Tallaugatta .	87 987 000 000	1894	12 581 000 000	1902	922 100	1894	4 100	Jan. 1903	2 400	The run-off during 1902 was 10% of the total rainfall. Gaugings from 1886.
Kiewa	Kiewa	29 506 000 000	1894	7 021 000 000	1902	159 000	1889	200	Feb. 1900	700	During 1902, 23% of the rainfall on the catchment area passed the gauging station. Gaugings from 1885.

Mr. Bellamy. the discharge for the twelve months only reached 1 024 000 000 cu. ft., of which a very small proportion passed Condobolin.

The Brisbane River has a total water-shed of about 5 300 sq. miles, the areas of the principal contributing rivers and creeks, respectively, being as follows:

Stanley River	600 sq. miles
Brisbane River, above Cooyar Creek	166 " "
Mousildale and Avoca Creeks	190 " "
Cooyar Creek	410 " "
Emu Creek	380 " "
Maroughi and Anduramba Creeks	180 " "
Cressbrook Creek	230 " "
Lockyer Creek	1 160 " "
Bremer River	780 " "
Remainder (about)	1 370 " "

This area is bounded on all sides by mountain ranges varying in height from 1 000 to 4 000 ft. More than 2 500 sq. miles in the upper portions of the water-shed consist generally of impervious strata, and the lower portions, or remainder, of permeable strata. The Stanley River, rising in high lands near the Pacific Coast, is subject to intense rainfall which has a quick run-off; indeed, so much so, that it is considered the chief factor in studying flood flows in the main river. The total length of the Brisbane River is 210 miles; it is tidal for 53 miles, and navigable for vessels of more than 12 000 tons as far as Brisbane. The tidal range at the mouth of the river and at Brisbane is between 3 and 8 ft.

The valley of the Brisbane River is the scene of recurring floods, and the highest flood on record at Brisbane occurred on February 5th, 1893. The rainfall recorded for 8 days previous to that date at meteorological stations within the water-shed is as follows:

Cressbrook	20.97 in.
Crohamhurst	83.43 "
Esk	18.60 "
Fassifern	4.53 "
Ipswich	10.49 "
Laidley	10.81 "
Nanango	6.14 "
Woodford	38.87 " (gauge overflowed)
Brisbane	19.42 "

Unfortunately, these records are insufficient to give any reliable data as to the true average fall over the whole area; they are given because they are correct. The cross-section at the railway crossing by Indooroopilly carried the whole of the flood-water. The estimated surface

velocity of the current on February 5th was 10 miles per hour. The discharge in the 24-hour period of the flood, when at its maximum height, may be taken at 34 500 000 000 gal. Mr. Bellamy.

The Fitzroy River drains a catchment area of about 58 000 sq. miles, the greater portion of which is very flat; and of this area not less than 54 900 sq. miles are above Rockhampton, the capital of Central Queensland. The extreme length of the river, including all bends, measured from the source of the Dawson River Branch to its outlet in Keppel Bay, is 520 miles. The length within tidal influence, extending up to Alligator Creek, about 29 miles above Rockhampton, is only 62 miles. The greatest flood on record occurred in February, 1896, and the estimated maximum discharge of the river for a 24-hour period was 397 226 000 000 gal. The lowest flow ever recorded was gauged by the writer in May, 1902, when the small quantity of 46 000 000 gal. was discharged in 24 hours. The physical conditions of this river are entirely different from those previously referred to, chiefly on account of the water-shed being so very flat.*

In conclusion, the writer desires to state that he considers Mr. Fuller's paper to be the best and most instructive contribution yet published on the subject.

E. Kuichling, M. Am. Soc. C. E. (by letter).—The author is entitled to unstinted credit for having performed a vast amount of useful work in preparing his extensive compilation and ingenious analysis of flood records of American rivers. The subject is, moreover, a timely one, in view of the extraordinary floods which have occurred this year in Ohio and New York, last year in Wisconsin, and three years ago in Europe. From his studies of the data submitted, he reaches the conclusion that a general formula for computing the probable maximum flood discharge from catchment areas, must be provided with a factor, or coefficient, C, the magnitude of which depends on the peculiarities of each water-shed, and is adapted thereto by considering all previous flood flows therefrom; and, furthermore, that it must have another factor to express the ratio of the probable future maximum discharge to past smaller maxima, which factor is $(1 + 0.8 \log. T)$, wherein T denotes the number of years in the period between the recurrence of floods of approximately the same magnitude. The general formula proposed by the author for the greatest average rate of flow during 24 hours, in cubic feet per second, is $Q = C\,A^{0.8}\,(1 + 0.8 \log. T)$, in which A is the area of the catchment basin, in square miles. Mr. Kuichling.

The values of the variable factor, C, for the rivers of the several geographical districts of the United States, adopted by the U. S. Geological Survey, are given as computed by the author in Tables 12 to 26, inclusive. Two sets of such factors are given, one referring to the

* "On the Rainfall of Central Queensland and Floods in the Fitzroy River," by Herbert E. Bellamy, *Minutes of Proceedings*, Inst. C. E., Vol. CLXIII, p. 295.

Mr. Kuichling. average yearly flood discharge in a series of years, T, of observation, and the other to the largest observed discharge during such period. For the sake of clearness, they should be designated, C_1 and C_2, their values being expressed by $C_1 = \frac{Q\,(\text{Ave.})}{A^{0.8}}$ and $C_2 = \frac{Q'}{A^{0.8}\,(1 + 0.8 \log. T)}$, in Columns 8 and 7, respectively, of those tables. It seems to be the author's purpose to use the value, C_1, in his aforesaid general formula for Q, as he places $Q = Q\,(\text{Ave.})\,(1 + 0.8 \log. T)$, thus giving $Q = C_1 A^{0.8} (1 + 0.8 \log. T)$. The use of the formula may be illustrated by the following example relating to the Susquehanna River at Binghamton, N. Y., in Table 14, where $A = 2\,400$, $T = 10$, $Q\,(\text{Ave.}) = 39\,100$, $C_1 = 78$, and $C_2 = 70$, corresponding to $Q' = 63\,000$ cu. ft. per sec., which is the largest flow observed in 10 years. If it be desired to compute the probable maximum discharge, Q, that will occur in a period of $T = 100$ years, at the same place, the formula will become: $Q = Q\,(\text{Ave.})\,(1 + 0.8 \log. T) = 39\,100 \times 2.6 = 101\,660$ cu. ft. per sec.; and in a period of $T = 1\,000$ years, it will be $Q = 39\,100 \times 3.4 = 132\,940$. The coefficient, C_2, should not be used, as it relates only to the particular values, Q' and $T = 10$.

An examination of Tables 12 to 26, inclusive, shows wide differences in the values of C_1 for apparently similar drainage areas. Thus, in Table 14, we have $C_1 = 50$ for the Passaic River, with $A = 823$ and $T = 34$; while for the Raritan River, with $A = 800$ and $T = 6$, the value of C_1 is 93. Similarly, in the same table, we find for the East and West Branches of the Delaware River, at Hancock, N. Y., $C_1 = 140$ and 105, respectively, for $A = 920$ and 680, and $T = 9$ in both cases. Again, in Table 13, we find $C_1 = 58$ and 49 for the Hudson River with, respectively, $A = 2\,800$ and $4\,500$, and $T = 13$ and 40; while for the Mohawk River, with $A = 3\,440$ and $T = 12$, we have $C_1 = 75$; also, for the neighboring water-sheds of West and East Canada Creeks, with $A = 364$ and 256, and $T = 9$ and 12, we find $C_1 = 116$ and 71, respectively. For nearly equal areas in the basins of the Connecticut, Mohawk, and Delaware Rivers, namely, $A = 3\,305$, $3\,440$, and $3\,250$, with $T = 11$, 12, and 8, we have $C_1 = 49$, 75, and 97, respectively; and similarly with many other catchment areas. The values of C_1 also vary at different points in the same river basin, sometimes increasing with A, sometimes being nearly constant, and sometimes decreasing.

The factor, C_1, appears to depend primarily on the depth and extent of the precipitation causing a flood, the season of the year, and the total drainage area at the point of observation; and, secondarily, on the nature of the surface soil of the water-shed, whether absorptive or impermeable, the slopes of the surface and lines of drainage, the shape of the basin and its component areas, the extent and character of the vegetation thereon, the duration of the excessive

rainfall and melting of previously fallen snow, and the extent to which a portion of the run-off is impounded in natural and artificial reservoirs, including the temporary inundation of broad flats in the valley above the point of observation. The latter is an important feature, and, in considering the probable future maximum flow from such catchment areas, it will be expedient to assume that improvements will be made, whereby inundations will be reduced. The factor is also affected by the formation and bursting of ice jams in northern streams, and the formation of barriers of sunken logs, silt, and gravel, which may cause a large storage temporarily until scoured away by a strong freshet. It is thus evident that numerous features must be taken into account in estimating the probable maximum flow from a large area. Mr. Kuichling

In regard to the rainfall, it can be said that heavy precipitations covering great areas of country during a few days, occur at more or less regular intervals of years in all the States east of the Mississippi River and on the Pacific Coast. The points where the rainfall is observed, however, are generally so far apart that it is very difficult to estimate the actual volume and distribution of the water on a large territory. In mountainous regions, intense precipitations often occur in localities not provided with rain-gauges, and the fact that unusual downpours have taken place on areas of many square miles, is attested by the resulting freshets, the magnitude of which is not warranted by the scanty available records of rainfall at other places. It happens, therefore, that the rainfall on mountainous and hilly catchment basins is frequently underestimated, as shown by the case of the water-shed of West Canada Creek, N. Y., where in one year the aggregate run-off was much more than the estimated precipitation.

In view of the limited periods of flood observation on most American rivers, and the recurrence of heavy rainfalls at longer intervals of time, it becomes questionable whether the use of different values of C_1 for similar water-sheds in the same region is proper. The same combination of conditions that produced a great flood in one year in a particular basin, is likely to occur in another year in the neighboring basin, and hence it seems safer to use the largest observed value of C_1 for all the streams of a given region when it is known that the area, topography, and character of soil are substantially alike. The variation in the value of C_1 for larger values of A of similar water-sheds of a region, can probably also be deduced from the data, whereby C_1 will be expressed as a function of A; and, as a result, we will have a formula like $Q = B\ A^n\ (1 + 0.8 \text{ log. } T)$ for all similar catchment basins in a particular region. By this means regional peculiarities would be recognized by variations in the value of the coefficient, B, and perhaps also of the exponent, n. The ad-

Mr. Kuichling.

vantage of such a formula lies in its applicability to a wide range in the value of A.

A question also arises as to the value of T to be used when computing the future maximum flood discharge. The tables contain no reference to the rainfalls that produced the floods listed, nor to the dates of their occurrence, and hence it is impracticable to determine from the data submitted whether a probable maximum flood did not occur during the period of observation. It may also be that the largest observed flood was nearly equal to the future maximum, in which event the factor $(1 + 0.8 \log. T)$ would be correspondingly smaller than 2.5 or 3.0, when T is taken at from 75 to 320 years. Much depends, therefore, on the actual conditions which produced the observed floods, and on this subject no information is given in the paper. The floods of some of the rivers in Ohio, in 1913, were unprecedented in magnitude, and a statement that they might become twice as large in the future would surely have to be accompanied with the most convincing proofs in order to be accepted; and the same can also be said of all other large floods elsewhere.

In regard to the intervals between the occurrence of extraordinary floods, few data are available for American rivers. Table 12 shows that two great floods of nearly equal magnitude occurred in the Connecticut River, at Hartford, Conn., in a period of 104 years; two in the Merrimac River, at Lawrence, Mass., in 56 years; and two in the Androscoggin River, at Rumford Falls, Me., in 40 years. Table 13 shows that one great flood was observed in the Hudson River, at Mechanicsville, N. Y., in 40 years; Table 14, that two great floods were observed in the Passaic River, at Dundee Dam, N. J., in 34 years; Table 17, that two great floods occurred in the Genesee River, at Rochester, N. Y., in 128 years; Table 16, that two great floods of the Ohio River, at Wheeling, W. Va., occurred in 50 years; and Table 20, that one large flood of the Kansas River, at Lecompton, Kans., occurred in 60 years, all other periods of observation being less than 30 years. The average of the foregoing enumeration is one great flood at intervals of 37 years, or a total of 14 in 512 years.

For foreign rivers, a few data as to recurrences of great floods were recently published in official investigations of floods of the Seine, at Paris, and the Danube, at Vienna.* In the Seine at Paris, the observations extend over a period of 400 years, the highest flood having occurred on March 1st, 1658, and the next almost equally high one on January 28th, 1910; the third in order of magnitude, and but slightly lower than the second, was on December 26th, 1740, and between this date and January 7th, 1883, eight other floods of somewhat lower height are recorded. We thus have for the Seine a record of 11 great floods

* Abstracts thereof are given in *Engineering News*, 1910, I, p. 327, and *Zeitschrift* des Oesterr. Ingenieur- u. Architekten-Vereines, 1910, pp. 147 and 457, respectively.

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in 255 years, or on the average one unusual flood in 23 years. The discharge of the river at Paris, on January 28th, 1910, was estimated at 83 500 cu. ft. per sec. from a water-shed of about 16 860 sq. miles.

At Vienna the drainage area of the Danube is about 39 200 sq. miles, and from well-attested flood marks, the highest flood occurred in 1501. No reliable data for computing the discharge at that time are available, but, as nearly as can be determined from observations of the river channel during the past 50 years, the maximum flow was then about 503 200 cu. ft. per sec. Numerous smaller floods have occurred since, with discharges reaching 370 800 cu. ft. per sec. in September, 1899; but from a careful study of all existing data relating to the precipitation on the drainage area, the engineer who reported on the subject in 1910 concluded that a recurrence of the great flood of 1501 was highly probable, and might take place in any year of excessive rainfall.

Other records for European rivers might also be cited, to show that great floods have occurred in shorter intervals than 100 years; but it is believed by the writer that enough has been adduced to show that the proper value of T to be used in making estimates of future flood flows of American rivers, deserves further explanation by the author.

Generous reference has been made in the paper to the writer's extensive study of the subject, as published in the State Engineer's "Report on the Barge Canal of the State of New York", Albany, 1901. In this publication the drainage areas and maximum flood discharges, in cubic feet per second per square mile, of 232 American and 364 foreign river basins were submitted, together with 18 different formulas, reprinted in Table 29, that had been devised up to 1900 by various engineers for estimating the probable maximum flow of a stream. Although these data are very useful, they are too long for reproduction here, especially as many of them are contained in the author's tables, and therefore the writer will append only the additional data on large flood discharges that he has collected since the year 1900.

An examination of the figures soon shows wide differences in the rates of maximum discharge for water-sheds of the same magnitude in different parts of the world, and confirms the view of the author that it is necessary to take into account the topographical, geological, and meteorological characteristics of a drainage area before estimating the probable maximum run-off therefrom. This opinion has long been held by hydrologists, and hence most of the various formulas for flood discharge are adapted only to particular localities. Doubtless the most important factors are the intensity, duration, and distribution of the rainfall on the water-shed, but owing to lack of sufficient data on this subject in almost every country and State, it becomes extremely difficult to establish even an approximately correct relation

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between them. For rains of comparatively long duration, the character of the surface soil seems to be of minor importance, as the ground usually becomes saturated in a few hours and absorption diminishes in large degree; whereas, for short heavy downpours, the condition of the soil and the vegetation thereon is of great influence on the run-off, especially as the area covered by such precipitation is then relatively small. The largest rate of run-off that the writer has found recorded is 3 200 cu. ft. per sec. per sq. mile, on July 14th, 1897, from an area of only 0.25 sq. mile of irregular rocky surface on Beacon Mountain, near Fishkill, N. Y.

Reference may also be made to the following formula* of R. Iszkowski, Chief Engineer of the Austrian Ministry of Public Works. This is called an "induction formula for estimating the normal and flood discharges, based on the characteristics of the water-shed," and involves four direct factors, viz.: the area (M) of the water-shed; the mean yearly depth (R) of rainfall thereon; two variable coefficients, one (C_1) depending on the topography or general slope of the territory, and the other (C_2) on the character of the surface soil, according as it is strongly absorptive, slightly permeable, or impervious; and lastly, a special factor (m) which varies inversely with the area of the water-shed. In its original form, for the metrical system, this formula is $Q_{max.} = (0.022\ C_1 + mC_2)\ R\ M$, where $Q_{max.}$ is the probable maximum flood discharge, in cubic meters per second, (R) is the mean annual depth of rainfall, in meters, and (M) is the drainage area, in square kilometers. For this system of measures, the value of the coefficient (C_1) ranges from 0.20 for very flat, sandy, or swampy areas, to 0.65 for high mountainous areas; the value of (C_2) ranges from 0.035 for very permeable land covered with vegetation to 0.70 for impervious rocky or frozen land, without active vegetation, and covered with snow which will increase the run-off by melting; and the value of (m) ranges from 7.88 for $M = 10$ sq. km. to 0.65 for $M = 100\,000$ sq. km., as set forth in a table from which the writer has deduced the approximate relation: $m = \dfrac{0.59\ (11\,050 + M)}{818 + M}$. For average conditions, such as correspond to a hilly territory with slightly permeable soil and sparse vegetation, the values of the said coefficients are $C_1 = 0.385$ and $C_2 = 0.40$. By substituting these particular values in the formula and then reducing to the customary measures of discharge (q) in cubic feet per second per square mile, area (M) in square miles, and the mean annual depth of rainfall (R) in inches, we will have:

$$q_{max.} = \frac{0.568\ .\ R\,(4\,129.5 + M)}{315.8 + M} \qquad \text{.................} (1)$$

* Published in *Wochenschrift* des Oesterr. Ingenieur- u. Architekten-Vereines, Vol. 9, 1884, pp. 25, 33, and 146.

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in cubic feet per second per square mile, for the aforesaid average conditions. If we assume $R = 36$ in., this expression will give $q_{max.} = 260, 208, 80$, and 28 for $M = 10, 100, 1\,000$, and $10\,000$, respectively. For mountainous territory with rocky or frozen soil, we will have $C_1 = 0.50$ and $C_2 = 0.60$ in the original formula, whence by reduction:

$$q_{max.} = \frac{0.848 \,.\, R\,(4\,147.4 + M)}{315.8 + M} \dots\dots\dots\dots\dots (2)$$

in cubic feet per second per square mile. For the same values of (R) and (M) as before, this second equation gives $q_{max} = 390, 312, 119$, and 42, or 50% more than by Equation (1). Similar expressions might also be deduced for other values of (C_1) and (C_2), but they will be omitted here as it is doubtful whether the formula can be applied generally without modification. In the writer's opinion, the formula gives values of (q) that are too low for small drainage areas and too high for large ones. The method of development, however, is ingenious and worthy of closer investigation.

On page 616 of the paper a formula is given that was devised by the writer in 1900 for the probable maximum flood discharge from mountainous and hilly water-sheds of not more than 5 000 sq. miles in the Middle and New England States. Since that time many other data have become available, so that this formula should now be modified. Further studies of the subject have led the writer to propose another simple formula which applies to river basins in the Southern Atlantic States, and is based on the greatest observed discharges of the Potomac River at Point of Rocks, Md., the New River at Radford, Va., the Catawba River at Rock Hill, N. C., the Little Tennessee River at Judson, N. C., Cane Creek at Bakersville, N. C., and numerous other streams which exhibit somewhat smaller rates of discharge than the preceding. This new formula is

$$q_{max.} = \frac{41.6\,(620 + M)}{24 + M} \dots\dots\dots\dots\dots\dots (3)$$

in cubic feet per second per square mile, and it may be regarded as applicable to mountainous and hilly water-sheds having areas of not more than 10 000 sq. miles, in the portion of the country indicated. In comparison with Iszkowski's data, as represented by the foregoing Equation (2), this formula gives $q_{max.} = 771, 242, 66$, and 44, respectively for $M = 10, 100, 1\,000$, and $10\,000$ sq. miles. A great difficulty is found in estimating the discharge from cloudbursts on basins of less than 50 sq. miles, as both the duration of the heavy rainfall and the area covered by it are indefinite; hence it is likely that a modification of this new formula will also be required.

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TABLE 37.—UNUSUAL FLOOD DISCHARGES, SUPPLEMENTARY TO THE DATA COMPILED BY THE WRITER, AND PUBLISHED IN THE REPORT ON THE PROPOSED BARGE CANAL FOR THE STATE OF NEW YORK, ALBANY, 1901, pp. 845-865. ARRANGED ACCORDING TO MAGNITUDE OF DRAINAGE AREA.

Stream and locality.	Drainage area, in square miles.	Maximum discharge, in cubic feet per second per square mile.	Date of flood.	Number of years observed.	Authority.*
AMERICAN STREAMS.					
Mississippi River, at St. Louis, Mo.	702 380	1.28	June, 1883	6 (1880-85)	(30)
Missouri River, at St. Charles, Mo.	530 810	1.13	June, 1883	6 (1880-85)	(30)
Missouri River, at Sioux City, Ia.	323 462	1.64	Apr., 1881	6 (1880-85)	(30)
Colorado River, at Yuma, Ariz.	225 000	0.67	June, 1909	11 (1902-12)	(6)
Ohio River, at Paducah, Ky.	205 750	7.00	Feb., 1884	6 (1880-85)	(30)
Mississippi River, at Grafton, Ill.	171 570	2.10	June, 1883	6 (1880-85)	(30)
Mississippi River, at Clayton, Ia.	79 040	2.66	June, 1880	6 (1880-85)	(30)
Kansas River, at Lawrence, Kans.	59 841	3.8	May 31, 1903	25 (1881-05)	(1)
Platte River, near Columbus, Nebr.	56 900	0.83	May 15, 1905	12 (1895-06)	(14)
Mississippi River, at Prescott, Wis.	44 070	2.50	Apr., 1881	6 (1880-85)	(30)
Colorado River, at Austin, Tex.	37 000	3.83	Apr. 7, 1900	9 (1896-04)	(1)
Mississippi River, at St. Paul, Minn.	36 085	3.32	Apr. 29, 1881	46 (1867-12)	(1) and (25)
Mississippi River, at St. Paul, Minn.	35 700	2.26	Apr., 1897	46 (1867-12)	(1) and (25)
Red River, at Grand Forks, N. D.	25 000	1.70	Apr., 1897	31 (1882-12)	(25)
North Platte River, at Camp Clarke, Nebr.	24 800	0.95	June 26, 1899	11 (1896-06)	(14)
Susquehanna River, at Harrisburg, Pa.	24 030	30.6	June, 1889	41 (1865-05)	(1)
Susquehanna River, at Harrisburg, Pa.	24 030	30.6	Mar., 1865	41 (1865-05)	(1)
Ohio River, at Wheeling, W. Va.	23 800	20.8	Feb. 7, 1884	22 (1884-05)	(1)
Ohio River, at Wheeling, W. Va.	23 800	19.00	Mar., 1907	28 (1884-11)	(2)
Republican River, at Bostwick, Nebr.	22 300	1.10	July 4, 1905	11 (1896-06)	(14)
Tennessee River, at Chattanooga, Tenn.	21 382	34.37	Mar. 11, 1867	38 (1867-04)	(1)
Ohio River, at Pittsburgh, Pa.	19 100	22.98	Mar. 15, 1907	30 (1884-13)	(33)
Mississippi River, at Anoka, Minn.	17 100	2.87	Apr., 1897	9 (1897 and 1905-12)	(25)
Illinois River, at Peoria, Ill.	15 700	3.66	Mar. 28, 1904	16 (1890-05)	(1)
Alabama River, at Selma, Ala.	15 400	9.5	Jan. 19, 1892	14 (1891-04)	(1)

* A list of these authorities is given at the end of this table.

TABLE 37.—(*Continued.*)

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Stream and locality.	Drainage area, in square miles.	Maximum discharge, in cubic feet per second per square mile.	Date of flood.	Number of years observed.	Authority.
Minnesota River, near Mankato, Minn.....	14 600	8.00	June, 1908	10 (1903-12)	(25)
Loup River, at Columbus, Nebr......	13 540	5.17	June 6, 1896	12 (1895-06)	(1)
Sacramento River, at Red Bluff, Cal........	10 400	24.42	Feb., 1909	11 (1902-12)	(8)
Sacramento River, at Jelly's Ferry, Cal.....	10 200	12.05	Mar., 1900	8 (1895-02)	(8)
Connecticut River, at Hartford, Conn.......	10 234	20.0	May, 1854	105 (1801-1905)	(1)
Susquehanna River, at Wilkes-Barre, Pa.....	9 810	22.2	May 2, 1902	7 (1899-05)	(1)
Potomac River, at Point of Rocks, Md.	9 654	48.9	June 2, 1889	18 (1889-06)	(1) and (17)
Potomac River, at Point of Rocks, Md.	9 654	22.66	Mar., 1902	18 (1889-06)	(1) and (17)
Blue River, near Manhattan, Kans..	9 490	7.25	May, 1903	9 (1895-03)	(19)
Allegheny River, at Kittanning, Pa........	9 010	26.66	Mar. 20, 1905	8 (1904-11)	(22)
Grand River, at Palisade, Colo........	8 546	4.88	June 5, 1905	4 (1902-05)	(1)
Smoky Hill River, at Ellsworth, Kans......	7 980	2.63	July, 1895	9 (1895-03)	(19)
Gunnison River, at Whitewater, Colo.....	7 863	3.67	June 5, 1905	4 (1902-05)	(1)
Penobscot River, at Bangor, Me.........	7 700	14.94	Apr. 10, 1901	35 (1875-09)	(10)
Savannah River, at Augusta, Ga..........	7 500	40.00	Sept. 11, 1888	66 (1840-05)	(1)
Delaware River, at Lambertville, N. J..	6 855	37.14	Jan. 8, 1841	120 (1786-1905)	(1)
Delaware River, at Lambertville, N. J..	6 855	32.62	June 8, 1862	120 (1786-1905)	(1)
Chippewa River, at Eau Claire, Wis........	6 740	9.00	June 8, 1905	4 (1902-05)	(1)
Cedar River, at Cedar Rapids, Ia......	6 320	3.75	Mar., 1910	3 (1909-11)	(4)
Rock River, below Rockton, Ill......	6 290	4.31	Mar., 1904	7 (1903-09)	(4)
Fox River, at Rapide Croche Dam, Wis.....................	6 200	2.49	June, 1895	10 (1895-04)	(18)
Niobrara River, near Valentine, Nebr....	6 070	1.15	July 18, 1903	7 (1901-07)	(14)
St. Croix River, near St. Croix Falls, Minn.................	5 930	5.65	May, 1912	7 (1902-05, 1910-12)	(25)
Monongahela River, at Lock No. 4, Pa.......	5 430	38.12	July 11, 1888	20 (1886-05)	(1)
Red Lake River, at Crookston, Minn.....	5 320	2.67	Apr., 1906	12 (1901-12)	(25)
Flint River, at Albany, Ga...........	5 000	7.79	Feb. 17, 1905	4 (1902-05)	(16)
Grand River, at Grand Rapids, Mich.	4 900	8.04	Mar. 27, 1904	Indefinite	(2)
Merrimac River, at Lawrence, Mass.....	4 553	18.04	Mar., 1896	59 (1846-04)	(15)
Mississippi River, above Sandy River, Minn.	4 510	2.12	Sept., 1900	18 (1895-12)	(25)

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TABLE 37.—(*Continued.*)

Stream and locality.	Drainage area, in square miles.	Maximum discharge, in cubic feet per second per square mile.	Date of flood.	Number of years observed.	Authority.
Hudson River, at Mechanicville, N. Y.	4 500	26.67	Mar. 28, 1913	26 (1888-13)	(37)
Kennebec River, at Waterville, Me.	4 270	35.36	Dec. 16, 1901	14 (1893-06)	(15)
Oconee River, at Dublin, Ga.	4 182	8.35	Feb. 12, 1903	8 (1898-05)	(16)
Pit River, near Bieber, Cal.	4 040	6.81	Mar., 1907	5 (1904-08)	(6)
Coosa River, at Rome, Ga.	4 006	16.02	Dec. 31, 1901	7 (1897-03)	(16)
Neosho River, at Iola, Kans.	3 670	20.33	July 10, 1904	9 (1896-04)	(2)
Feather River, at Oroville, Cal.	3 640	51.37	Mar., 1907	11 (1902-12)	(6)
Crow Wing River, near mouth, Minn.	3 580	2.85	Apr., 1897	3 (1882, 1884 and 1897)	(25)
Mohawk River, at Cohoes, N. Y.	3 472	28.50	Mar. 27, 1913	26 (1888-13)	(37)
Chattahoochee River, at West Point, Ga.	3 300	26.86	Dec. 30, 1901	10 (1896-05)	(16)
Wabash River, at Logansport, Ind.	3 163	17.99	Mar. 27, 1904	21 (1885-05)	(2)
Klamath River, at Keno, Ore.	3 150	2.68	June, 1904	7 (1904-10)	(6)
Link River, at Klamath Falls, Ore.	3 110	2.90	Mar., 1904	7 (1904-10)	(6)
Verdigris River, at Liberty, Kans.	3 067	16.45	July 8, 1904	10 (1895-04)	(2)
Shenandoah River, at Millville, W. Va.	2 995	46.65	Oct., 1896	12 (1895-06)	(17)
Catawba River, near Rock Hill, S. C.	2 987	50.50	May 23, 1901	9 (1895-03)	(34)
Hudson River, at Glens Falls, N. Y.	2 760	25.36	Mar. 28, 1913	15 (1899-13)	(37)
Saline River, at Beverly, Kans.	2 730	5.86	June, 1896	9 (1895-03)	(19)
New River, at Radford, Va.	2 725	63.78 63.37	Oct., 1900 May, 1901	13 (1898-10)	(21*a*)
Savannah River, near Calhoun Falls, S. C.	2 712	27.76	Feb. 14, 1900	6 (1896-1900 & 1903)	(16)
Kennebec River, bet. Forks & Waterville.	2 700	48.56	Dec. 16, 1901	14 (1893-06)	(15)
Wisconsin River, near Merrill, Wis.	2 630	8.02	Sept. 16, 1903	4 (1902-05)	(18)
Sangamon River, at Riverton, Ill.	2 560	7.50	Oct., 1911	4 (1908-11)	(4)
Elkhorn River, near Norfolk, Nebr.	2 470	3.24	May 30, 1903	8 (1896-03)	(14)
Chemung River, at Chemung, N. Y.	2 440	21.52	Mar. 28, 1913	11 (1903-13)	(37)
Ocmulgee River, at Macon, Ga.	2 425	20.97	Mar. 1, 1902	13 (1893-05)	(16)
Menominee River, near Iron Mountain, Mich.	2 415	4.87	May, 1904	4 (1902-05)	(18)
Genesee River, at Rochester, N. Y.	2 365	17.42	Mar. 28, 1913	22 (1892-13)	(37)
Kern River, at Bakersfield, Cal.	2 345	4.05	June, 1906	19 (1893-11)	(7)

TABLE 37.—(*Continued.*)

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Stream and locality.	Drainage area, in square miles.	Maximum discharge, in cubic feet per second per square mile.	Date of flood.	Number of years observed.	Authority.
Androscoggin River, at Rumford Falls, Me....	2 320	23.81	Apr. 22, 1895	12 (1893-04)	(15)
N. Fork Feather River, at Big Bend, Cal........	1 940	56.34	Mar. 19, 1907	6 (1905-10)	(8)
American River, at Fair Oaks, Cal........	1 910	55.00	Mar., 1907	9 (1904-12)	(8)
W. Branch Penobscot River, at Millinocket, Me.	1 880	12.90	Apr. 1, 1903	9 (1901-09)	(10)
Kings River, near Sanger, Cal	1 740	25.25	Jan., 1901	18 (1895-12)	(7)
Kiskiminetas River, at Avonmore, Pa........	1 720	39.10	Mar., 1908	5 (1907-11)	(22)
Allegheny River, at Red House, N. Y......	1 640	25.00	Mar. 2, 1910	8 (1904-11)	(22)
San Joaquin River, at Hamptonville, Cal..	1 637	36.53	Jan., 1881	16 (1879-82) (1895-01) and (1907-12)	(7)
Oostanaula River, at Resaca, Ga..........	1 614	14.88	Mar. 17, 1899	7 (1896-1901 and 1905)	(16)
Kennebec River, at Forks, Me............	1 570	12.67	Dec. 16, 1901	6 (1901-06)	(15)
S. Fork Shenandoah River near Front Royal, Va....	1 570	48.92	Mar., 1902	8 (1899-06)	(17)
Chattahoochee River, at Oakdale, Ga	1 560	31.28	Dec. 30, 1901	10 (1895-04)	(16)
Catawba River, at Catawba, N. C........	1 535	61.89	May 23, 1901	10 (1896-05)	(34)
Tuolumne River, at Lagrange, Cal........	1 500	35.07	Jan., 1911	18 (1895-12)	(7)
S. Branch Potomac River, near Springfield, W. Va.................	1 440	17.81	Mar., 1906	8 (1894-5) (1899-01) (1904-06)	(17)
Genesee River, at Mt. Morris, N. Y......	1 410	12.52	Mar. 27, 1913	22 (1892-13)	(37)
Cheat River, at Morgantown, W. Va..	1 380	30.29	Jan., 1911	13 (1899-11)	(22)
Tygart Valley River, at Felterman, W. Va.....	1 327	26.36	Jan., 1911	5 (1907-11)	(22)
Youghiogheny River, at Connellsville, Pa......	1 320	27.50	June, 1910	4 (1908-11)	(22)
Chagres River, at Gatun, Panama.......	1 320	93.9	Dec. 28, 1909	20 (1894-13)	(32)
Mohawk River, at Little Falls, N. Y.....	1 306	26.65	Mar. 27, 1913	16 (1898-13)	(37)
Cache Creek, at Yolo, Cal............	1 230	16.34	Feb., 1909	10 (1903-12)	(8)
Yuba River, near Smartsville, Cal....	1 220	90.91	Jan., 1909	10 (1903-12)	(8)
Raquette River, at Massena Springs N.Y.	1 170	9.40	May, 1909	8 (1904-11)	(5)
E. Branch Penobscot River, at Grindstone, Me.	1 100	23.36	Sept. 29, 1909	8 (1902-09)	(10)
Merced River, near Merced Falls, Cal..	1 090	34.13	Jan., 1911	11 (1901-11)	(7)
French Creek, at Carlton, Pa..........	1 070	22.93	Mar., 1910	4 (1908-11)	(22)
Sacandaga River, at Hadley, N. Y.........	1 060	27.36	Mar. 28, 1913	7 (1907-13)	(37)

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TABLE 37.—(*Continued.*)

Stream and locality.	Drainage area, in square miles.	Maximum discharge, in cubic feet per second per square mile.	Date of flood.	Number of years observed.	Authority.
Scioto River, at Columbus, O.	1 047	80.82	Mar. 25, 1913	Indefinite.	(39)
N. Fork Shenandoah River, near Riverton, Va.	1 037	20.86	Apr., 1901	8 (1899-06)	(17)
Flint River, near Woodbury, Ga.	988	30.62	Feb. 28, 1902	6 (1900-05)	(16)
Truckee River, near State line, Cal.	955	16.02	Mar., 1907	14 (1899-12)	(6)
Stanislaus River, at Knight's Ferry, Cal.	935	61.18	Mar., 1907	10 (1903-12)	(7)
Clarion River, at Clarion, Pa.	910	43.20	Mar., 1905	27 (1885-11)	(22)
Schoharie Creek, at Fort Hunter, N. Y.	909.3	44.56	Mar. 27, 1913	16 (1898-13)	(37)
Schoharie Creek, at Fort Hunter, N. Y.	900.0	55.11	Mar. 21, 1901	16 (1898-13)	(37)
Youghiogheny River, below Confluence, Pa.	874	52.63	Aug. 21, 1888	32 (1874-05)	(1)
Dead River, near The Forks, Me.	870	20.74	May, 1904	5 (1902-06)	(15)
Minnesota River, above Whetstone River, Minn.	846	0.11	Mar., 1903	6 (1899-04)	(25)
Kettle River, near Sandstone, Minn.	825	7.15	May, 1912	4 (1909-12)	(25)
Passaic River, at Dundee, N. J.	823	38.16	Oct. 10, 1903	95 (1810-05)	(24)
Raritan River, at Bound Brook, N. J.	806	64.52	Sept. 24, 1882	96 (1810-05)	(1)
Putah Creek, at Winters, Cal.	805	37.27	Mar., 1907	8 (1905-12)	(8)
North River, at Port Republic, Va.	804	29.69	Sept., 1896	5 (1895-99)	(17)
Hudson River, at North Creek, N. Y.	804	35.08	Mar. 28, 1913	7 (1907-13)	(37)
Chagres River, at Bohio, Panama	779	115.5	Dec. 27, 1909	20 (1894-13)	(32)
Broad River, near Carlton, Ga.	762	38.22	Feb. 28, 1902	9 (1897-05)	(16)
West Fork River, at Enterprise, W. Va.	744	23.67	Jan., 1911	5 (1907-11)	(22)
Big Muddy River, near Cambon, Ill.	735	14.97	May, 1911	4 (1908-11)	(4)
Santa Ynez River, near Lompoc, Cal.	725	28.14	Mar., 1911	5 { (1906-08) (1910-12) }	(6)
Raquette River, at Piercefield, N. Y.	723	8.13	May, 1911	4 (1908-11)	(5)
Little Tennessee River, at Judson, N. C.	675	85.3	Dec., 1901	15 (1896-10)	(21)
Monocacy River, near Frederick, Md.	660	31.00	Mar., 1902	11 (1896-06)	(17)
Mokelumne River, near Clements, Cal.	642	26.01	Jan., 1911	8 (1905-12)	(7)
McCloud River, near Gregory, Cal.	608	68.26	Mar., 1904	7 (1902-08)	(8)
Hoosic River, at Johnsonville, N. Y.	605	38.01	Mar. 28, 1913	11 (1903-13)	(37)
Etowah River, at Canton, Ga.	604	28.30	Dec. 29, 1901	9 (1896-04)	(16)

TABLE 37.—(*Continued.*)

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Stream and locality.	Drainage area, in square miles.	Maximum discharge, in cubic feet per second per square mile.	Date of flood.	Number of years observed.	Authority.
Stony Creek River, near Fruto, Cal.	601	48.75	Feb., 1909	12 (1901-12)	(8)
Tugaloo River, near Madison, S. C.	593	36.86	July 1, 1905	8 (1898-05)	(16)
Conewango Creek, at Frewsburg, N. Y.	550	20.95	Jan., 1911	2 (1910-11)	(22)
Santa Catarina River, at Monterey, Mex.	544	590.00	Aug. 27, 1909	Indefinite.	(35)
Coosawattee River, at Carters, Ga.	531	31.92	May 21, 1901	10 (1896-05)	(16)
Cosumnes River, at Michigan Bar, Cal.	524	42.75	Jan., 1911	6 (1907-12)	(7)
Truckee River, at Tahoe, Cal.	519	2.60	July, 1907	14 (1895 and 1900-12)	(6)
Olentangy River, at Columbus, O.	514	70.00	Mar. 25, 1913	Indefinite.	(39)
N. Fork Feather River, below Prattville, Cal.	506	19.47	Mar., 1907	6 (1905-10)	(8)
Deerfield River, at Shelburne Falls, Mass.	501	42.51	Apr. 15, 1909	4 (1907-10)	(12)
Cache Creek, at Lower Lake, Cal.	500	8.68	Feb., 1909	12 (1901-12)	(8)
Ausable River, at Ausable Forks, N. Y.	487	45.17	Mar. 27, 1913	....	(37)
Cattaraugus Creek, at Versailles, N. Y.	467	53.53	Mar. 25, 1913	....	(37)
Tionesta Creek, at Nebraska, Pa.	451	20.40	Jan., 1911	2 (1910-11)	(22)
Casselman River, at Confluence, Pa.	448	43.89	Mar., 1907	7 (1905-11)	(22)
Battenkill, at Greenwich, N. Y.	444	21.65	Mar. 28, 1913	3 (1911-13)	(37)
Whetstone River, at Bigstone, S. D.	441	2.95	Apr., 1910	9 (1899-04) (1910-12)	(25)
Apalachee River, near Buckhead, Ga.	440	15.19	Mar. 1, 1902	5 (1901-05)	(16)
Youghiogheny River, at Confluence, Pa.	435	52.07	Mar., 1907	7 (1905-11)	(22)
Chagres River, at Alhajuela, Panama	427	398.1	Dec. 26, 1909	20 (1894-13)	(32)
S. Fork Sangamon River, at Taylorville, Ill.	427	9.70	Sept., 1911	4 (1908-11)	(4)
Rio Mora, at Weber, N. M.	423	65.70	Sept. 29, 1904	....	(2)
Mahoning Creek, at Furnace Bridge, Pa.	412	30.51	Feb., 1910	2 (1910-11)	(22)
Hiwassee River, at Murphy, N. C.	410	54.54	Mar. 19, 1899	9 (1897-05)	(16)
N. Branch Potomac River, at Piedmont, W. Va.	410	32.80	Feb., 1902	8 (1899-06)	(17)
Tygart Valley River, at Belington, W. Va.	403	40.88	July, 1907	5 (1907-11)	(22)
Mohave River, at Victorville, Cal.	400	33.58	Mar., 1908	7 (1899-05)	(6)
Pacolet River, at Spartansburg, S. C.	400	88.90	June 6, 1903	....	(19)
Calaveras River, at Jenny Lind, Cal.	395	176.20	Jan., 1911	6 (1907-12)	(7)
Middle Oconee River, near Athens, Ga.	395	49.52	Feb. 28, 1902	2 (1901-02)	(16)

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TABLE 37.—(*Continued.*)

Stream and locality.	Drainage area, in square miles.	Maximum discharge, in cubic feet per second per square mile.	Date of flood.	Number of years observed.	Authority.
Black Lick Creek, at Black Lick, Pa.	386	50.82	Mar., 1905	8 (1904-11)	(22)
E. Fork Carson River, near Gardnerville, Nev.	381	8.69	Feb., 1904	10 { 1890-91 / 1900-10 }	(6)
Pompton River, at Two Bridges, N. J.	380	61.60	Oct. 10, 1903	....	(1)
Rondout Creek, at Rosendale, N. Y.	380	51.34	Apr. 26, 1910	12 (1901-12)	(9)
Esopus Creek, at Mt. Marion, N. Y.	378	65.34	Apr. 26, 1910	6 (1907-12)	(9)
W. Canada Creek, at Trenton Falls, N. Y.	376	96.54	Dec. 15, 1901	16 (1898-13)	(37)
W. Canada Creek, at Trenton Falls, N. Y.	376	69.15	Mar. 28, 1913	16 (1898-13)	(37)
W. Canada Creek, at Hinckley, N. Y.	372	104.57	Apr. 21, 1869	45 (1869-13)	(37)
Carrabassett River, at N. Anson, Me.	340	40.21	May, 1904	5 (1902-06)	(15)
Silver Creek, near Lebanon, Ill.	335	15.64	May, 1908	4 (1908-11)	(4)
San Luis Rey River, near Pala, Cal.	318	40.88	Mar., 1906	8 (1903-11)	(6)
Oil Creek, at Rouseville, Pa.	302	27.71	Mar., 1910	2 (1910-11)	(22)
Antietam Creek, near Sharpsburg, Md.	295	23.17	Feb., 1902	9 (1897-05)	(17)
Youghiogheny River, at Friendsville, Md.	294	27.76	Mar., 1904	6 (1899-04)	(22)
Brokenstraw Creek, at Youngsville, Pa.	290	24.50	Mar., 1910	2 (1910-11)	(22)
Piscataquis River, at Foxcroft, Me.	286	77.62	Sept. 29, 1909	8 (1902-09)	(10)
Crooked Creek, at Hileman's Farm, Pa.	279	43.37	Sept., 1911	2 (1910-11)	(22)
Nottely River, at Ranger, N. C.	272	20.81	Feb. 28, 1902	5 (1901-05)	(16)
Miller Creek, near Lovella, Ore.	270	24.93	Feb., 1907	9 (1904-12)	(6)
Tule River, near Portersville, Cal.	266	20.41	Dec., 1909	12 (1901-12)	(7)
Bear River, at Van Trent, Cal.	263	98.10	Mar., 1907	9 (1904-12)	(8)
Salmon River, at Pulaski, N. Y.	260	41.65	Mar. 27, 1913	16 (1898-1913)	(37)
Cahokia Creek, near Poag, Ill.	259	13.90	Oct., 1911	3 (1909-11)	(4)
East Canada Creek, at Dolgeville, N. Y.	256	54.30	Mar. 27, 1913	16 (1898-13)	(37)
Susan River, at Susanville, Cal.	256	7.03	Mar., 1903	6 (1900-05)	(6)
South River, at Port Republic, Va.	246	37.40	Sept., 1896	5 (1895-99)	(17)
Cobbosseecontee Stream, at Gardiner, Me.	240	13.65	Mar., 1903	17 (1890-06)	(15)
Esopus Creek, at Olivebridge, N. Y.	239	64.39	Apr. 26, 1910	7 (1906-12)	(9)
Toccoa River, near Blueridge, Ga.	231	53.20	Aug. 23, 1901	6 (1898-03)	(16)
Alcovy River, near Covington, Ga.	228	9.52	Feb. 28, 1902	4 (1901-04)	(16)

TABLE 37.—(*Continued.*)

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Stream and locality.	Drainage area, in square miles.	Maximum discharge, in cubic feet per second per square mile.	Date of flood.	Number of years observed.	Authority.
San Gabriel River, near Azusa, Cal.	222	56.31	Jan., 1910	19 (1894-12)	(6)
Sapella River, near Los Alamos, N. M.	221	36.67	Sept. 29, 1904	....	(2)
Arroyo Seco, near Soledad, Cal.	215	61.86	Mar., 1911	12 (1901-12)	(6)
N. Branch French Creek, at Kimmeytown, Pa.	212	43.85	Jan., 1911	2 (1910-11)	(22)
Catskill Creek, at S. Cairo, N. Y.	210	100.00	Spring, 1901	....	(27)
Santa Ynez River, near Santa Barbara, Cal.	207	45.65	Jan., 1907	6 (1904-07) (1911-12)	(6)
Fishkill Creek, at Glenham, N. Y.	198	69.19	Mar. 1. 1902	3 (1901-03)	(21)
Tallulah River, at Tallulah Falls, Ga.	191	40.63	Dec. 29. 1901	4 (1900-01) (1904-05)	(16)
Santa Ana River, near Mentone, Cal.	182	26.97	Apr., 1903	11 (1902-12)	(6)
E. Branch Fish Creek, at Taberg, N. Y.	169	65.09	Mar. 27, 1913	16 (1898-13)	(37)
Kinzua Creek, at Dewdrop, Pa.	162	19.94	Mar., 1910	2 (1910-11)	(22)
Ramapo River, at Pompton, N. J.	160	65.88	Sept. 22, 1882	....	(24)
Rio Mora, below Mora, N. M.	159	139.70	Sept. 29, 1904	....	(2)
Turtle Creek, at East Pittsburgh, Pa.	146	64.21	Mar., 1904	Indefinite.	(26)
Oriskany Creek, at Oriskany, N. Y.	144	51.0	Dec. 16, 1901	7 (1898-04)	(27)
Devil's Creek, near Viele, Ia.	143	1 300.0	June 10, 1905	....	(1)
Santa Ysabel Creek, near Escondido, Cal.	128	50.78	Jan., 1909	7 (1906-12)	(6)
Laurel Hill Creek, at Confluence, Pa.	126	40.00	Mar., 1907	7 (1905-11)	(22)
Rockaway River, at Boonton, N. J.	118	48.85	Oct. 10, 1903	....	(24)
Lewistown Reservoir, Outlet, Ohio	111	57.66	Mar. 25, 1913	Indefinite.	(38)
Onondaga Creek, at Syracuse. N. Y.	108	30.00	Mar. 25, 1913	Indefinite.	(37)
E. Branch Fish Creek, at Point Rock, N. Y.	104.3	80.54	Fall, 1897	Indefinite.	(27)
Little Stony Creek, near Lodoga. Cal.	102	69.22	Feb., 1909	5 (1908-12)	(8)
Wanaque River, at Pompton, N. J.	101	83.61	Oct. 10, 1910	Indefinite.	(24)
Putah Creek, near Guenoc, Cal.	91	198.90	Mar., 1904	3 (1904-06)	(8)
Butte Creek, near Butte Valley, Cal.	73	22.47	Jan., 1909	6 (1905-10)	(8)
Loramie Reservoir Outlet, Ohio	72	97.22	Mar. 25, 1913	Indefinite.	(38)
W. Fork Carson River, at Woodfords, Cal.	70	22.43	May, 1906	12 (1900-11)	(6)
Pequannock River, at Macopin, N. J.	62	90.84	Oct. 10, 1903	Indefinite.	(20)

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TABLE 37.—(*Continued.*)

Stream and locality.	Drainage area, in square miles.	Maximum discharge, in cubic feet per second per square mile.	Date of flood.	Number of years observed.	Authority.
N. Fork Cottonwood Creek, at Ono, Cal.	52	77.70	Feb., 1909	6 (1907-12)	(8)
Kosk Creek, near Henderson, Cal.	51.9	44.32	Apr., 1911	2 (1910-11)	(8)
Six Mile Creek, at Ithaca, N. Y.	46.0	185.0	June 21, 1905	Indefinite.	(1)
Elkhorn Creek, at Keystone, W. Va.	44.0	1 363.0	June 22, 1901	Indefinite.	(34)
Basic Creek, at Freehold, N. Y.	41.0	81.22	Spring, 1901	Indefinite.	(27)
Whippany River, at Whippany, N. J.	38.0	84.20	Feb. 6, 1896	Indefinite.	(24)
Bear Grass Creek, at Louisville, Ky.	27.5	100.0	Feb. 22, 1908	Indefinite.	(40)
Pequonnock River, near Bridgeport, Conn.	25.0	157.0	July 29, 1905	Indefinite.	(1)
Pinal Creek, at Globe, Ariz.	25.0	560.	Aug. 17, 1904	Indefinite.	(2)
Cane Creek, at Bakersville, N. C.	22.0	1 341.	May 20, 1901	Indefinite.	(34)
Willow Creek, near Heppner, Ore.	20.0	1 800.	June 14, 1903	Indefinite.	(19)
Goodyear Creek, at Goodyear Bar, Cal.	12.2	96.72	Jan. 30, 1911	Indefinite.	(8)
Mill Brook, at Sherburne, N. Y.	9.4	241.0	Sept. 4, 1905	Indefinite.	(1)
Camp Branch, at Ensley, Ala.	7.43	68.77	June, 1909	2 (1909-10)	(11)
Mill Brook, at Sherburne, N. Y.	5.0	262.0	Sept. 4, 1905	Indefinite.	(1)
Reel's Creek, near Deerfield, N. Y.	4.42	66.92	June 21, 1903	4 (1901-04)	(28)
Venison Branch, near Mulga, Ala.	3.87	53.49	June, 1909	2 (1909-10)	(11)
Estanzuela River, near Monterey, Mex.	3.50	825.0	Aug. 28, 1909	Indefinite.	(35)
Starch Factory Creek, near New Hartford, N. Y.	3.40	151.6	July 11, 1905	3 (1903-05)	(29)
Rio Grande, near Culebra, Panama.	2.36	161.0	May 25, 1911	Indefinite.	(32)
Cherryvale Creek, at Cherryvale, Kans.	2.00	930.0	....	Indefinite.	(31)
Budlong Creek, near Utica, N. Y.	1.13	120.40	Mar. 25, 1904	Indefinite.	(2)
Beacon Brook, near Fishkill, N. Y.	0.25	3 200.00	July 14, 1897	18 (1896-13)	(27)
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Elbe River, at Altengamm, Germany	60 600	2.15	....	Indefinite.	(55)
Danube River, at Vienna, Austria.	39 212	8.86	July and August, 1897	Indefinite.	(50)
Po River, at Ponte Lagosiuro, (Italy)	27 027	9.10	....	Indefinite.	(53)
Elbe River, at Torgau, Saxony.	22 040	6.73	....	Indefinite.	(55)
Elbe River, at Tetschen, Saxony.	19 711	7.98	Sept., 1890	Indefinite.	(50)

TABLE 37.—(*Continued.*)

Stream and locality.	Drainage area, in square miles.	Maximum discharge, in cubic feet per second per square mile.	Date of flood.	Number of years observed.	Authority.
Seine River, at Paris, France.........	16 859	5.24	Jan. 28, 1910	Indefinite.	(51)
Weser River, at Baden, Germany...	14 640	11.10 10.08	Jan., 1841 Mar., 1881	Indefinite. Indefinite.	(41)
Moldau River, at Prague, Austria......	10 404	13.48	Sept., 1890	Indefinite.	(50)
Main River, at Frankfurt, Germany..............	9 570	13.00 9.85 9.84	Jan., 1342 Mar. 31, 1845 Nov. 27, 1882	570	(57)
Moselle River, at Trier, Germany.......	9 200	17.28	....	Indefinite.	(47)
Weser River, at Hoya, Germany.......	8 615	12.30	Jan., 1841	Indefinite.	(41)
Loire River, at junction with the Allier, France........	6 945	26.44	——, 1846	Indefinite.	(62)
Weser River, at Hameln, Germany....	6 475	15.01	Jan., 1841	Indefinite.	(41)
Aller River, at Westen, Germany....	5 881	10.51	....	Indefinite.	(41)
Weser River, below the Diemel at Karlshafen, Germany.	5 730	15.71	Jan., 1841	Indefinite.	(41)
Durance River, at Bonpas, France.....	5 714	37.08 37.04	Nov. 11, 1886 ——, 1843	7 (1882-88) Indefinite.	(44) (62)
Glommen River, at Elverum, Norway....	5 650	15.63	....	42 (1871-12)	(60)
Allier River, at junction with the Loire, France..........	5 548	30.00	——, 1856	Indefinite.	(62)
Aller River, below junction with the Leine River, Germany.	5 302	11.32	....	Indefinite.	(41)
Little Elbe River, at Brandeis, Germany...	5 052	5.04	Sept., 1890	Indefinite.	(50)
Weser River, above the Diemel at Karlshafen, Germany.	5 050	16.44	Jan., 1841	Indefinite.	(41)
Segura River, in Eastern Spain........	4 830	10.8	....	Indefinite.	(22)
Durance River, at Mirabeau, France	4 533	52.20	Nov. 11, 1886	Indefinite.	(61)
Allier River, at Nevers, France.......	4 500	37.3	....	Indefinite.	(22)
Vistula River, at Cracow, Galicia.......	3 180	34.7	——, 1813	Indefinite.	(55)
Ems River, below Meppen, Germany.	3 173	8.43	Dec., 1880	Indefinite.	(41)
Mur River, at Graz, Austria.........	2 950	13.0	....	Indefinite.	(55)
Aller River, above junction with the Leine River, Germany.	2 788	10.45	....	Indefinite.	(41)
Fulda River, at Münden, Germany..	2 686	25.64	Jan. 18, 1841	Indefinite.	(41)
Fulda River, at Kassel, Germany.....	2 600	26.49	Jan. 18, 1841	Indefinite.	(41)
Rhine River, at entrance to Lake Constance, Switzerland...	2 555	48.4	....	Indefinite.	(55)

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TABLE 37.—(*Continued.*)

Stream and locality.	Drainage area, in square miles.	Maximum discharge, in cubic feet per second per square mile.	Date of flood.	Number of years observed.	Authority.
Leine River, above junction with the Aller River, Germany.	2 514	12.29	....	Indefinite.	(41)
Leine River, at Hanover, Germany.	2 022	12.76 16.25	Mar., 1881 —, 1808	Indefinite. Indefinite.	(41) (41)
Ems River, at Meppen, Germany....	1 963	9.83	Dec., 1880	Indefinite.	(41)
Rhone River, at St. Maurice, Switzerland...............	1 812	12.47 17.40	—, 1897 —, 1856	10 (1890-99) Indefinite.	(44) (44)
Glatzer Neisse River, at junction with the Oder, Germany........	1 759	24.70	July 26, 1903	Indefinite.	(44)
Bober River, at Sagan, Silesia, Germany..................	1 640	43.7	July 31, 1897	Indefinite.	(46)
Oder River, at Sagan, Silesia........	1 638	43.1	July 31, 1897	Indefinite.	(58)
Moselle River, at Toul, France..........	1 430	19.77	Oct. 23, 1880	Indefinite.	(47)
San River, Galicia, Austria	1 420	64.6	—, 1867	Indefinite.	(55)
Eder River, at junction with the Fulda, Germany.........	1 298	35.37	Jan., 1841	Indefinite.	(41)
Hase River, at Meppen, Germany....	1 210	6.16	Dec., 1880	Indefinite.	(41)
Verdon River, at junction with the Durance, France..........	932	62.90	Nov. 1, 1843	Indefinite.	(61)
Glatzer Neisse River, Silesia, Germany........	906	46.7	....	Indefinite.	(22)
Ardeche River, at junction with the Rhone, France.........	831	382.48	—, 1827	Indefinite.	(63)
Eder River, at Felsberg, Germany...	708	8.64	....	Indefinite.	(41)
Dreimel River, at Karlshafen, Germany.	681	40.45	Nov., 1890	Indefinite.	(41)
Werre River, at mouth, Germany...	575	48.14 32.18	Feb., 1799 Nov., 1890	Indefinite. Indefinite.	(41) (41)
Eder River, at Hemfurt, Germany...	552	58.0	....	Indefinite.	(22)
Buech River, at junction with the Durance, France......	552	84.57	Nov. 1, 1843	Indefinite.	(61)
Schwalm River, at junction with the Eder, Germany........	498	18.42	Jan., 1841	Indefinite.	(41)
Innerste River, at mouth, Germany...	477	15.53 24.74	Mar., 1881 —, 1808	Indefinite. Indefinite.	(41) (41)
Bober River, near Mauer, Silesia......	467	90.8	July, 1897	Indefinite.	(59)
Pegnitz River, at Nuremberg, Germany.	459	33.1	Feb. 6, 1909	Indefinite.	(56) and (52)
Orne River, at Caen, France........	449	19.67	Oct., 1880	Indefinite.	(62)
Oker River, near Braunschweig, Germany............	416	22.50	Mar. 11, 1881 and July 12, 1898	Indefinite.	(41)

TABLE 37.—(*Continued.*)

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Stream and locality.	Drainage area, in square miles.	Maximum discharge, in cubic feet per second per square mile.	Date of flood.	Number of years observed.	Authority.
Malapane River, at proposed dam site, Germany	403	26.3	....	Indefinite.	(22)
Hotzenplotz River, at junction with the Oder, Germany	392	76.94	July 21, 1903	Indefinite.	(44)
Ubaye River, at junction with the Durance, France	361	127.12	Nov. 1, 1843	Indefinite.	(61)
Coulon River, at junction with the Durance, France	352	100.33	Nov. 1, 1843	Indefinite.	(61)
Bleone River, at junction with the Durance, France	351	115.71	Nov. 1, 1843	Indefinite.	(61)
Sill River, at Innsbruck, Austria	330	9.63	....	Indefinite.	(44)
Moselle River, at Epinal, France	313	90.45	....	Indefinite.	(47)
Asse River, at junct. with the Durance, France	285	111.52	Nov. 1, 1843	Indefinite.	(61)
Wupper River at mouth, Westphalia, Germany	240	90.0	....	Indefinite.	(22)
Werre River, at Herford, Germany	238	13.80	Feb., 1799	Indefinite.	(41)
Wien River, at Vienna, Austria	216	81.3	Mar. 23, 1883	Indefinite.	(55)
Ardeche River, at Vans, France	215	525.63	——, 1890	Indefinite.	(63)
Bober River, at Rohrlach, Germany	204.6	160.1	July 30, 1897	Indefinite.	(48)
Queis River, at Lauban, Germany	187.4	161.2	July 30, 1897	Indefinite.	(46)
Ardeche River, at Aubenas, France	178.0	694.41	——, 1890	Indefinite.	(63)
Aupa River, near Slatina, Bohemia	158.6	73.46	....	Indefinite.	(22)
Weisseritz River, Saxony, Germany	148.0	69.0	July 31, 1897	Indefinite.	(48)
Bega River, near mouth, Germany	147.0	15.20	Jan., 1881	Indefinite.	(41)
Urft River, near Heimbach, Westphalia	145.0	24.7	Before 1897	Indefinite.	(48)
	145.0	24.4	——, 1909	Indefinite.	(22)
Dreimel River, at Marsberg, Germany	131.0	65.80	Nov., 1890	Indefinite.	(41)
Wupper River, at Barmen, Germany	122.0	88.37	Nov. 24, 1890	Indefinite.	(46)
Queis River, at Marklissa, Germany	118.0	262.7	Aug. 3, 1888	Indefinite.	(49)
	118.0	233.3	July 30, 1897	Indefinite.	(46)
Jaispitz Creek, at Weirowitz, Austria	90.7	31.0	Mar., 1888	Indefinite.	(54)
Wupper River, at Dahlhausen, Germany	82.3	88.2	Nov. 24, 1890	Indefinite.	(48)
Queis River, at Greiffenberg, Germany	78.0	172.0	July 30, 1897	Indefinite.	(48)

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TABLE 37.—(*Continued.*)

Stream and locality.	Drainage area, in square miles.	Maximum discharge, in cubic feet per second per square mile.	Date of flood.	Number of years observed.	Authority.
Wilga Creek, near Cracow, Galicia....	52.1	65.1	....	Indefinite.	(55)
Holsterwitz Creek, at Gr. Olkowitz, Austria.	36.3	25.3	Mar., 1888	Indefinite	(54)
Bargaglino Creek, at Genoa, Italy..........	35.6	485	Oct., 1892	Indefinite.	(52)
Bargaglino Creek, at Genoa, Italy..........	35.6	421	July 18, 1908	Indefinite.	(52)
Eyach River, at Balingen, Wurtemberg..................	34.75	356.0	June 5, 1895	Indefinite.	(53)
Urnäsch River, St. Gallen, Switzerland..	30.0	153.03	....	Indefinite.	(44)
Goldbach, at Arnoldsdorf, Germany..............	19.7	268.88	July 21, 1903	Indefinite.	(44)
Queis River, near head, Germany....	12.34	358.0	July 31, 1897	Indefinite.	(58)
Little Aupa River, near head, Germany.....	11.9	385.63	....	Indefinite.	(22)
Furens River, at St. Etienne, France...	9.65	478.0	——, 1849	Indefinite.	(64)
Bargaglino Creek, above Genoa, Italy......	8.8	732.0	Oct., 1892	Indefinite.	(52)
Eyach River, near Margarethausen, Wurtemberg..........	7.34	788.8	June 5, 1895	Indefinite.	(53)
Dabrowka Creek, near Sambor, Austria...	5.02	253.2	....	Indefinite.	(55)
Eschbach, at Remscheid, Germany.	1.74	92.00	....	Indefinite.	(44)
Alfeldbach, at dam site, Alsace......	1.62	208.0	....	Indefinite.	(45)

List of Authorities Referred to in Table 37.

I. References to American Streams.

Water Supply Papers of the United States Geological Survey, Washington, D. C., as follows:

(1) Paper No. 162, published in 1906. (2) No. 147, published in 1905. (3) No. 311, published in 1912. (4) No. 305, published in 1912. (5) No. 304, published in 1912. (6) No. 300, published in 1913. (7) No. 299, published in 1912. (8) No. 298, published in 1912. (9) No. 281, published in 1912. (10) No. 279, published in 1912. (11) No. 262, published in 1911. (12) No. 261, published in 1911. (13) No. 260, published in 1911. (14) No. 230, published in 1909. (15) No. 198, published in 1907. (16) No. 197, published in 1907. (17) No. 192, published in 1907. (18) No. 156, published in 1906. (19) No. 96, published in 1904. (20) No. 92, published in 1904. (21) No. 75, pub-

lished in 1903. (21*a*) Virginia Geol. Survey, Hydrography, 1906. (22) Pittsburgh Flood Com. Report, 1912. (23) N. J. Geol. Survey, Report for 1894. (24) N. J. Geol. Survey, Report for 1903. (25) Minnesota State Drainage Com. Report for 1912. (26) Water Supply Com. of Pennsylvania, Report for 1908. (27) Report of N. Y. State Engineer for 1902. (28) Report of N. Y. State Engineer for 1904. (29) Report of N. Y. State Engineer for 1905. (30) Report on Reservoirs in Wyoming and Colorado, Washington, D. C., 1898. (31) *Transactions*, Am. Soc. C. E., 1905, Vol. LIV, p. 200. (32) *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 871. (33) *Proceedings*, Engineers' Society of Western Pennsylvania, Vol. 23 (1907), pp. 306-418. (34) *Engineering News*, 1902, II, p. 104. (35) *Engineering News*, 1909, II, p. 315. (36) *Engineering News*, 1913, I, p. 672. (37) *Engineering Record*, 1913, I, p. 399. (38) *Engineering Record*, 1913, I, p. 440. (39) *Engineering Record*, 1913, I, pp. 444 and 592. (40) L. Metcalf, M. Am. Soc. C. E. Mr. Kuichling.

II. Reference to European Streams.

(41) *Die Weser und Ems*, Berlin, 1901, Vols. 2, 3, and 4. (42) *Handbuch der Ing'rwiss'n, Wasserbau*, Vol. 3, Leipzig, 1900. (43) *Handbuch der Ing'rwiss'n, Wasserbau*, Vol. 6, Leipzig, 1910. (44) *Handbuch der Ing'rwiss'n, Wasserbau*, Vol. 13, Leipzig, 1908, p. 189. (45) *Talsperrenbau*, P. Ziegler, 2d Ed., 1911, p. 316. (46) *Ztsch. Ver. Deutscher Ingenieure*, Vol. 50 (1906), Articles by Prof. Intze. (47) *Ztsch. für Gewässerkunde*, Vol. 8 (1908). (48) *Ztsch. für Arch. u. Ingenieurwesen*, Vol. 45 (1899), p. 7. (49) *Zentralblatt der Bauverwaltung*, 1889, p. 80. (50) *Zentralblatt der Bauverwaltung*, 1899, p. 9. (51) *Zentralblatt der Bauverwaltung*, 1910, p. 113. (52) *Zentralblatt der Bauverwaltung*, 1911, pp. 80 and 180. (53) *Deutsche Bauzeitung*, 1898, p. 62. (54) *Landwirtschaftliche Jahrbücher*, Vol. 28, 1899. (55) *Wochensch. d. Oester. Ing'r u. Arch. Vereines*, Vol. 9 (1884), pp. 33, 136, and 146. (56) *Ztsch. d. Oester. Ing'r u. Arch. Vereines*, 1911, p. 381. (57) *Das Städtische Tiefbauwesen in Frankfurt*, 1903. (58) *Engineering News*, 1911, II, p. 683. (59) *Engineering News*, 1913, I, p. 672. (60) *Engineering Record*, 1913, II, p. 583. (61) *Annales des Ponts et Chaussées*, 1892, 1st Sem., p. 1. (62) *Annales des Ponts et Chaussées*, 1897, 3d Trim. (63) *Annales des Ponts et Chaussées*, 1904, 3d Trim., p. 130. (64) E. Wegmann, Design of Dams, N. Y., 1911, p. 65.

Robert E. Horton, M. Am. Soc. C. E. (by letter).—On page 610, Mr. Fuller states that: Mr. Horton.

"As far as the writer knows, the first suggestion of the use of the element of time in a formula for flood flows was made by Allen Hazen, M. Am. Soc. C. E., in a memorandum written by him in 1910."

It is not disputed that the idea of the use of the element of time in the discussion of flood flows may have occurred to Mr. Hazen independently, but in deference to the late George W. Rafter, M. Am. Soc. C. E., the writer feels obliged to deny that the idea originated with Mr. Hazen. This idea, as to its application to floods and also to other hydrological data, was suggested to the writer at least as early as 1896, or 14 years before it was suggested to Mr. Fuller by Mr. Hazen.

Mr. Horton. It was Mr. Rafter's desire that the writer should prepare the data for a paper on the application of the Theory of Probability to Flood Flows and other hydrological data, and the matter was discussed frequently and fully during the years 1896 to 1904. It appeared at that time, however, that stream flow data were so meager as to make it desirable to defer the completion of the studies. The writer continued the studies, as occasion arose, using the ordinary law of probability where the data were too meager to permit of plotting a satisfactory special probability curve on logarithmic paper.

In 1896 the writer had prepared, at Mr. Rafter's request (by letter dated October 25th, 1896), an analysis of the rainfall and run-off of the Upper Hudson Drainage Basin, in which the probable average interval of recurrence of run-off less than the observed minimum was determined by the Theory of Probability. This and similar phenomena were analyzed by the ordinary method of probability, through the use of the so-called Gaussian Law of Error.*

It was recognized at that time that the Gaussian Law did not represent accurately the law of frequency of recurrence of many hydrological events, and an effort was made first to find a better general law† and to deduce individual laws. For the latter, the method of plotting flood records on logarithmic paper was tried, and proved so promising that, between 1900 and 1908, many logarithmic probability curves were plotted by the writer and corresponding formulas deduced, involving time as an element; and the methods and results have been applied extensively in professional practice by the writer for a period of several years.

He also applied the method of analysis of flood data by logarithmic special probability curves to the determination of maximum navigable stages, capacity of waste-weirs, etc., in connection with his work as Resident Engineer in Charge of the Bureau of Hydraulics of the New York State Barge Canal.

These results, after discussion with other engineers of the Department and with the Advisory Board of Consulting Engineers, were used as a basis in the design of work costing some millions of dollars. In this work the writer adopted 100 years in most cases as the average interval of recurrence of a flood of the magnitude to be made the basis of design. For the determination of relative frequencies of floods of different magnitudes on small torrential drainage basins in the Upper Mohawk River area, for which flood records were mostly wanting, the writer used as a basis of comparison in 1906 logarithmic flood dis-

* Report of State Engineer and Surveyor of New York, 1896, p. 841 *et seq.*

† In a letter dated August 11th, 1897, Mansfield Merriman, M. Am. Soc. C. E., suggested to the writer the use of a function of the form $y = (x + a)^2 (x - b)^2$ for this purpose.

charge formulas which he had previously deduced for the Neshaminy, Tohickon and Perkiomen, as follows: Mr. Horton.

$$Q = 30\ T^{0.25} \text{ for the Neshaminy,}$$
$$Q = 30\ T^{0.27} \text{ " " Perkiomen,}$$
$$Q = 40\ T^{0.25} \text{ " " Tohickon,}$$

in which Q = flood discharge equalled or exceeded in an average interval of T years, in cubic feet per second per square mile.

These formulas were based on 20 years' records of the streams, covering the period, 1885-94. They give the results shown in Table 38.

The writer also deduced the following general formula, applicable to these and similar drainage basins:

$$Q = 4\,021.5 \frac{T^{\frac{1}{4}}}{A},\ T = \sqrt[4]{\frac{Q\ A}{4\,021.5}}$$

or, as a logarithmic formula,

$$\log. Q = 3.6043881 + \frac{1}{4} \log. T - \log. A,$$

in which A = drainage area, in square miles; other notations as in Table 38.

TABLE 38.

Stream.	Drainage area, in square miles.	Interval, T, in Years.				
		1	10	25	50	100
		Cubic feet per second per square mile.				
Perkiomen	152	29.7	49.6	60.5	70.5	82
Neshaminy	139.3	29.4	55.0	71.0	85.5	104
Tohickon	102.2	40.5	71.0	89.0	105.0	125

In view of the foregoing considerations, the writer fails to discover wherein there is any proper basis for a claim of originality for this method, as far as the use of time as an element in flood formulas is concerned, as set forth in the paper. The writer does not claim any great breadth of applicability for the general formula just given, but believes that factors other than area modify the flood discharge of streams so profoundly that it is better, wherever possible, to derive individual formulas or flood-frequency diagrams for each stream. Following this idea, he deduced flood-frequency diagrams in 1906-08, for use in connection with the New York Barge Canal work, for the Mohawk, Hudson, Oswego, Seneca, and other streams. He has deduced and used similar special probability curves and logarithmic formulas to determine the law of frequency of recurrence of various other hydro-

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logical events, such as minimum seasonal rainfall, duration of periods without rain, relative probability of floods in different months of the year, minimum run-off of streams, storage volume required to maintain a constant supply from a stream, rain intensity of various durations, etc.

Attention may properly be called to another study, utilizing the same principles, which seems to antedate Mr. Hazen's suggestion of 1910, namely, the rain intensity diagram and formulas of J. de Bruyn-Kops, M. Am. Soc. C. E., for Savannah, Ga.*

With reference to the Neshaminy, Perkiomen, and Tohickon flood formulas derived by the writer in 1904, as given in Table 88, it should be noted that these were derived in such a manner as to include all floods exceeding a given magnitude, and not merely the greatest single day's flood of each year. The writer's usual method of deriving a formula for the frequency of annual floods which will equal or exceed a given limit is similar to that described in Table 5, Part *c*, except that in his analysis Mr. Fuller discards all floods less than the mean, and works from ratios. The writer utilizes all annual floods, both greater than and less than the mean, although it is sometimes necessary to use a different flood formula for floods of low intensity. He also prefers to work directly from the original data rather than by reducing the data to the form of ratios.

Mr. Fuller says of his general formula, as given on page 567.

"$Q = CA^{0.8}$ $(1 + 0.8 \log. T)$, in which Q is the largest 24-hour average rate of flow to be expected in a period of T years."

Tracing out the derivation of this formula (given on pages 576-580), it appears that Q is there defined as the "average maximum flood," and the meaning of this term appears to be quite different from that first given.

Assuming that Mr. Fuller's formula represents the average magnitude of floods having an average interval of recurrence greater than T, it may also be said that it represents the magnitude of a flood which will be equalled or exceeded at some average interval greater than T, but that this interval is unknown and cannot be determined from his formula. This illustrates the indefiniteness of his formulas when applied to the problem of flood determination, in the form in which, from the writer's experience, this problem usually occurs.

Again, in the case of Tohickon Creek, Mr. Fuller apparently does not mean by his formula for the average maximum flood of that stream, $Q = Q$ (Ave.) $(1 + 0.76 \log. T)$, that we may expect a flood at least as great as, or in other words, equal to or greater than 6 300 cu. ft. per sec. once in 5 years, for his own analysis of the same data (Table 5, Part *c*) shows that we may expect a flood to equal or exceed

* *Transactions*, Am. Soc. C. E., Vol. LX, p. 248.

6 300 cu. ft. per sec. on an average only once in about 10 years, or one-half as often. **Mr. Horton.**

On reading the paper carefully, the writer is of the opinion that the author has failed to set forth clearly the meaning of his own deductions. The writer's understanding of the matter is as follows, referring to Table 5, Part *a*, for Tohickon Creek.

The average magnitude of maximum floods having average intervals of recurrence between 5 and 25 years is 1.53 times the mean flood, or 6 300 cu. ft. per sec.

The average magnitude of floods having intervals of recurrence between 2.50 and 25 years is 1.29 times the mean flood, or 5 310 cu. ft. per sec., and so on.

The results of the author's general formula bear the same interpretation, except that it is probably approximately true, from the method of analysis used, that Q, in the formula, represents approximately the average volume of all floods having an average interval of recurrence greater than T, it being here assumed that the formula is correct for the particular stream in question.

What Mr. Fuller has really deduced is not the average magnitude of the floods having, for example, 25 years or 12.5 years average interval of occurrence. Statistically speaking, there can be only one magnitude of the maximum flood having, for example, 25 or 12.5 years average interval of recurrence. The appellation "average maximum flood" of a given interval of recurrence seems to be misleading and inaccurate. If we had a long enough record, and were to divide it up arbitrarily into 25-year periods, and were to take the average of the maximum floods of each period, we would get a quantity having some semblance to a maximum average flood of 25 years interval, but the writer fails to see any practical value in such a result. Furthermore, the result would obviously be very likely to be affected by the mode of dividing up the record, which condemns the method.

Take, for example, the 5-year Tohickon flood of 6 290 cu. ft. per sec. The author does not mean that we may expect a flood of just this magnitude on an average of once in 5 years. As a matter of fact, the probable average interval of recurrence of a flood of just this magnitude is very much greater than once in 5 years.

The writer cannot see that results in the form of the "average maximum flood" are of much practical use, but, if any one wants them, they are derived more easily and directly by the method shown in Table 5, Part *c*, than by the method used by the author.

In addition, it would seem that the method of Table 5, Part *c*, when properly applied, has several other decided advantages.

1.—It is simpler of application, requiring three columns instead of five.

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2.—It gives directly the average interval of recurrence of a flood equal to or greater than a given magnitude. This is a factor of great importance, and has direct application in designing spillways, determining high navigable stages of waterways, etc.

3.—It enables the average interval of recurrence of a flood lying between any two given magnitudes to be readily determined.

For example, from Table 5, Part *c*, the average interval of Tohickon floods at least 1.21 times the mean, or 4 968 cu. ft. per sec., is five years. Their probability is, therefore, $\frac{1}{5} = \frac{5}{25}$.

The average interval of floods of at least 1.45 times the mean, or 5 958 cu. ft. per sec., is 8.33 years. Their probability is, therefore, $\frac{1}{8.33} = \frac{3}{25}$. The probability of a flood between 4 968 and 5 958 cu. ft. per sec. is, therefore, $\frac{5}{25} - \frac{3}{25} = \frac{2}{25} = 0.080$, or the probable average interval of recurrence of a flood between 4 968 and 5 958 cu. ft. per sec., or of a flood having an average of about 5 460 cu. ft. per sec., is 12.33 years. Now, if Table 5, Part *a*, and the formulas deduced by the method there given, mean what the author says they do, would we not expect floods of this average magnitude to occur at intervals of about 3.57 years, instead of 12.33 years, as actual experience shows to be the case? This illustrates what seems to be a fatal error in the author's method of analysis.

It would appear that the relation between flood magnitude and frequency, when expressed in terms of either "average maximum flood" or "median flood," is not only very indefinite and difficult of comprehension, but is apt to be very misleading, if it is not indeed practically meaningless. It does not directly convey the information which the engineer usually most desires, for example: If a spillway has a capacity of 1 000 cu. ft. per sec., how often on an average will its capacity be exceeded?

Further, in Tables 4 and 5 for Tohickon Creek, the author has arbitrarily excluded from consideration all floods which did not exceed the average yearly one of 4 117 cu. ft. per sec. He has excluded thirteen floods and used eleven. Instead of following the rule "of making the best possible use of all the available information", he has made no direct use of considerably more than half of it. What the author was seeking was, it may be assumed, to find the law defining the relation between the magnitude of floods and their relative frequency of recurrence. If one were seeking to determine the law of velocity of falling bodies, and had 25 observations of the time required for different weights, varying from 10 to 60 lb., to fall through given heights, it would hardly be considered good scientific method to discard *a priori* all observations on weights of less than 35 lb.

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Mr. Fuller's method of working with ratios (Table 5), instead of actual flood volumes, seems to be confusing and cumbersome. The data which the engineer has to start with are actual flood volumes; what he desires to determine, as a rule, as in designing a spillway, is a flood volume which will not be equalled or exceeded on an average oftener than once in a given interval, say, 100 years.

If we had two drainage basins, A and B, which were similar figures, with similar drainage systems, or "stream nets" as the German hydrologist calls them, and identical in geology, soil, topography, slope. culture, and climate, but if A were larger than B, and if Q_A and Q_B were the true maximum 24-hour floods to be expected on these streams in an average interval of T years, then the difference between Q_A and Q_B would be the difference in flood-yielding capacity due to area alone.

In practice, however, it often happens that one area much larger than another does not yield a materially larger flood. If the data of such heterogeneous basins are plotted together in terms of area as abscissas, it appears to the writer that the results will be so discordant as to render the conclusions drawn therefrom of doubtful value.

Here we come squarely against the question of the physical, and not merely the statistical, aspects of the question. The latter only are covered by Mr. Fuller's discussion.

In the climate of the North Atlantic States local thunder storms nearly always produce maximum floods on very small areas, though only broad general cyclonic storms produce floods on the larger and medium large rivers, like the Hudson, the Susquehanna, and the Delaware. Now, thunder storms produce rains of great intensity but short duration. Broad cyclonic storms produce rains of low intensity but long duration, so that the relative flood intensity for different areas is not wholly determined by the relative times required for the water to reach the stream, or by the physiographic or cultural conditions, but partly by purely meteorological conditions.

The paper contains a valuable collection of flood data. It would have been easy to include the dates of the maximum floods, and these would be of use to engineers in comparing simultaneous flood intensities at different places. The writer cannot help feeling a sense of regret that so much labor has been expended in analysis of flood data along lines that do not seem to be either the most scientific or the most useful to engineers.

The ambition to place the results of one's investigations in the hands of the Profession at the earliest date is laudable, if indulged in with moderation. The writer's only excuse for withholding from publication the results of his own studies along similar lines for so long a time is that he thought it better to try the method out in prac-

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tice first, rather than run the chance of punishing the Profession by the publication of that which might later prove to be premature.

The writer happens to have collected all the data contained in Table 13. Many of the floods there listed have subsequently been exceeded.*

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G. B. Pillsbury, Assoc. M. Am. Soc. C. E. (by letter).—Accepting the premise that the magnitude of a flood is due to the combination of an unlimited number of accidental causes, the probability of the occurrence of a flood exceeding a given magnitude should follow the well-established laws of probability as deduced in the theory of errors of observations. Under these laws the probability of an annual flood less than a given value, Q', is given by the formula:

$$P = \frac{h}{\sqrt{\pi}} \int_0^k e^{-h^2 x^2} dx \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

and of an annual flood greater than Q'

$$P = \frac{h}{\sqrt{\pi}} \int_k^\infty e^{-h^2 x^2} dx = \frac{1}{\sqrt{\pi}} \int_t^\infty e^{-t^2} dt \ldots\ldots (2)$$

where $k = Q' - Q$ (Ave.) and $t = h\,k$.

The value of h is given by the formula:

$$h = \sqrt{\frac{n-1}{2 \Sigma v^2}}$$

where n is the number of floods observed and Σv^2 is the sum of the squares of the residuals of the observed values with relation to the average yearly flood.

The reciprocal of P, Equation 2, will then represent the period of years in which, in the long run, an annual flood equalling or exceeding Q' may be expected.

The usual probability tables give the value of the function, $\frac{2}{\sqrt{\pi}} \int_0^t e^{-t^2} dt$, from which the function, $\frac{1}{\sqrt{\pi}} \int_t^\infty e^{-t^2} dt$, in Equation 2 can be computed from the relation,

$$\frac{1}{\sqrt{\pi}} \int_t^\infty e^{-t^2} dt = \frac{1}{2} \left(1 - \frac{2}{\sqrt{\pi}} \int_0^t e^{-t^2} dt\right).$$

The author, however, has taken the average value of the annual floods equalling or exceeding the given value as his measure of the

* Reports on Hydrography (by the writer), State Engineer's Reports, 1900, 1912; also paper on "Effects of Recent Flood on New York Streams," *Engineering Record*, April 12th, 1913.

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expected flood, and not the flood of magnitude which would be derived from Equation 2.

The mathematical expression for this average or mean of floods exceeding Q' is

$$Q = Q\,(\text{Ave.}) + \frac{\dfrac{h}{\sqrt{\pi}} \displaystyle\int_k^{\infty} x\, e^{-h^2 x^2}\, dx}{\dfrac{h}{\sqrt{\pi}} \displaystyle\int_k^{\infty} e^{-h^2 x^2}\, dx}$$

$$= Q\,(\text{Ave.}) + \frac{1}{h}\;\frac{\dfrac{1}{2\sqrt{\pi}}\, e^{-t^2}}{\dfrac{1}{\sqrt{\pi}} \displaystyle\int_t^{\infty} e^{-t^2}\, dt} \quad \ldots\ldots(3)$$

$$= Q\,(\text{Ave.}) + \frac{1}{h}\, M \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(4)$$

It is obvious that this mean value is somewhat greater than its inferior limit.

Table 39 shows the relation between the time, T, considered (or the reciprocal of P in Equation 2), and the value of M (Equation 4).

TABLE 39.

t	$T = \frac{1}{P}$	M
0.0	2.	0.564
0.2	2.57	0.697
0.4	3.50	0.841
0.6	5.05	0.994
0.8	7.77	1.155
1.0	12.72	1.32
1.2	22.82	1.49
1.4	42.02	1.67
1.6	84.88	1.85
1.8	185.2	2.05
2.0	434.8	2.25
2.2	1 000.	2.34

Taking the floods of Tohickon Creek, as given by the author in Table 4, the computation of the mathematical values of average maximum flood, under the theory of probabilities, is shown in Table 40.

The probable average maximum floods, according to the theory of probabilities, and the observed averages as determined by the author, are plotted on Fig. 14.

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Table 41 and Fig. 15 show the floods of the Connecticut River at Hartford for 70 years, beginning 1843. The observed average maximum floods shown on the figure are computed in Table 42.

The floods of the Allegheny River at Kittanning for a 41-year period are tabulated in Mr. Knowles' discussion, Table 35. The value of h, as derived from these data is 2.03. The curve of probable average maxima, the author's curve, and the observed values, are plotted in Fig. 16.

TABLE 40.

Date.	Annual flood.	Ratio to average.	V	V^2
1884	4 379	1.06	+ 0.06	0.0036
5	3 664	0.89	− 0.11	0.0121
6	5 359	1.30	+ 0.30	0.0900
7	2 544	0.62	− 0.38	0.1444
8	3 493	0.85	− 0.15	0.0225
9	4 714	1.15	+ 0.15	0.0225
1890	2 942	0.71	− 0.29	0.0841
1	2 858	0.69	− 0.31	0.0961
2	3 158	0.76	− 0.24	0.0576
3	2 994	0.73	− 0.27	0.0729
4	8 650	2.10	+ 1.10	1.2100
5	3 857	0.93	− 0.07	0.0049
6	6 515	1.59	+ 0.59	0.3461
7	3 683	0.89	− 0.11	0.0121
8	4 160	1.01	+ 0.01	0.0001
9	3 222	0.78	− 0.22	0.0484
1901	4 089	0.99	− 0.01	0.0001
2	5 958	1.45	+ 0.45	0.2025
3	4 968	1.21	+ 0.21	0.0441
4	4 395	1.06	+ 0.06	0.0036
5	4 175	1.01	+ 0.01	0.0001
6	3 200	0.78	− 0.22	0.0484
7	4 120	1.00	0	0
8	2 770	0.67	− 0.33	0.1089
9	3 050	0.74	− 0.26	0.0676
Average = 4 117		$\Sigma v^2 = 2.7027$	$h = 2.11$.	

A very cursory examination of the figures shows that, for individual rivers, the observed average maxima differ widely, both from the author's formula and from their theoretical values under the theory of probability. This, indeed, is not extraordinary. The highest of the plotted observed figures is but a single flood, and not an average. The first few of the succeeding observed averages are based on very limited data, and may be expected to differ from their theoretical values in the long run. In the case of Tohickon Creek, the extraordinarily large value of the maximum observed flood has a very great bearing in the large values of the next lower averages.

The application of the law of probability to the maximum average floods, as given by the writer, is intended merely as a side light on the

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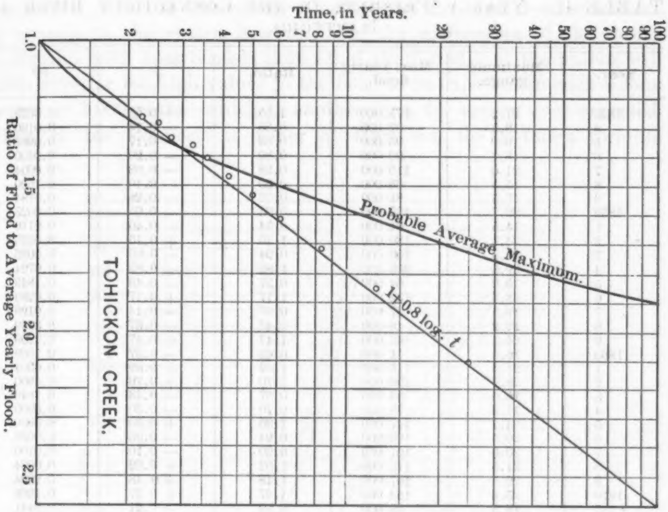

Fig. 14.

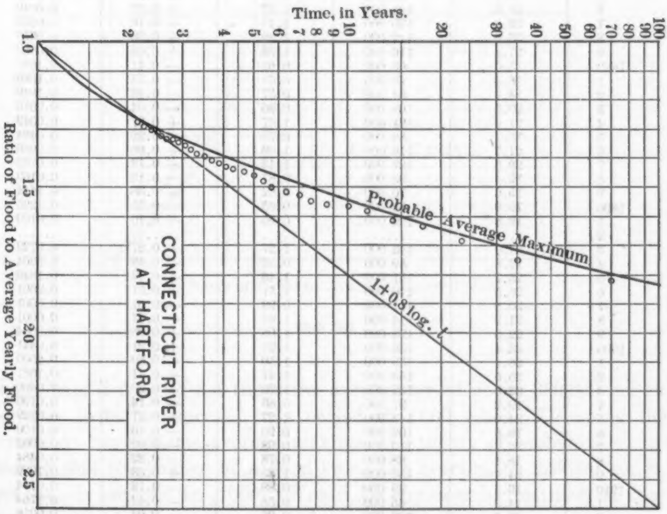

Fig. 15.

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TABLE 41.—YEARLY FRESHETS IN THE CONNECTICUT RIVER AT HARTFORD.

Year.	Maximum gauge.	Max. yearly flood.	Ratio.	V	V^2
1843	27.2	175 000	1.55	+ 0.55	0.3025
4	19.5	97 000	0.86	− 0.14	0.0196
5	19.0	93 000	0.83	− 0.17	0.0289
6	18.7	90 000	0.80	− 0.20	0.0400
7	21.0	110 000	0.98	− 0.02	0.0004
8	15.5	68 000	0.60	− 0.40	0.1600
9	17.5	81 000	0.72	− 0.28	0.0784
1850	20.7	107 000	0.95	− 0.05	0.0025
1	14.5	61 000	0 54	− 0.46	0.2116
2	23.1	130 000	1.15	+ 0.15	0.0225
3	20.5	106 000	0.94	− 0.06	0.0036
4	29.8	205 000	1.82	+ 0.82	0.6724
5	15.0	64 000	0.57	− 0.43	0.1849
6	23.3	132 000	1.17	+ 0.17	0.0289
7	19.5	97 000	0.86	− 0.14	0.0196
8	12.2	48 000	0.43	− 0.57	0.3249
9	26.4	166 000	1.47	+ 0.47	0.2209
1860	16.0	71 000	0.63	− 0.37	0.1369
1	21.5	115 000	1.02	+ 0.02	0.0004
2	28.7	192 000	1.70	+ 0.70	0.4900
3	15.0	64 000	0.57	− 0.43	0.1849
4	17.2	79 000	0.70	− 0.30	0.0900
5	24.7	147 000	1.30	+ 0.30	0.0900
6	20.5	106 000	0.94	− 0.06	0.0036
7	20.0	101 000	0.90	− 0.10	0.0100
8	21.5	115 000	1.02	+ 0.02	0.0004
9	26.5	167 000	1.48	+ 0.48	0.2304
1870	25.3	154 000	1.37	+ 0.37	0.1369
1	18.5	89 000	0.79	− 0.21	0.0441
2	19.7	99 000	0.88	− 0.12	0.0144
3	20.9	109 000	0.97	− 0.03	0.0009
4	23.8	137 000	1.22	+ 0.22	0.0484
5	18.4	90 000	0.80	− 0.20	0.0400
6	21.9	121 000	1.07	+ 0.07	0.0049
7	22.8	130 000	1.15	+ 0.15	0.0225
8	23.9	142 000	1.26	+ 0.26	0.0676
9	21.4	116 000	1.03	+ 0.03	0.0009
1880	15.0	66 000	0.59	− 0.41	0.1681
1	16.4	75 000	0.67	− 0.33	0.1089
2	14.7	64 000	0.57	− 0.43	0.1849
3	20.5	108 000	0.96	− 0.04	0.0016
4	21.9	121 000	1.07	+ 0.07	0.0049
5	18.1	88 000	0.78	− 0.22	0.0484
6	21.7	119 000	1.06	+ 0.06	0.0036
7	22.5	127 000	1.13	+ 0.13	0.0169
8	19.4	98 000	0.87	− 0.13	0.0169
9	15.6	70 000	0.62	− 0.38	0.1444
1890	16.0	73 000	0.65	− 0.35	0.1225
1	19.8	102 000	0.90	− 0.10	0.0100
2	....	....	....	......	......
3	24.0	143 000	1.27	+ 0.27	0.0729
4	13.8	59 000	0.52	− 0.48	0.2304
5	25.7	161 000	1.43	+ 0.43	0.1849
6	26.5	170 000	1.51	+ 0.51	0.2601
7	20.8	111 000	0.99	− 0.01	0.0001
8	21.2	114 000	1.01	+ 0.01	0.0001
9	22.0	122 000	1.08	+ 0.08	0.0064
1900	23.4	136 000	1.21	+ 0.21	0.0441
1	26.4	169 000	1.50	+ 0.50	0.2500
2	25.5	159 000	1.41	+ 0.41	0.1681
3	23.3	135 000	1.20	+ 0.20	0.0400
4	19.2	97 000	0.86	− 0.14	0.0196
5	24.0	143 000	1.27	+ 0.27	0.0729
6	19.8	102 000	0.90	− 0.10	0.0100
7	20.7	110 000	0.98	− 0.02	0.0004
8	18.1	88 000	0.78	− 0.22	0.0484
9	24.7	150 000	1.33	+ 0.33	0.1089
1910	19.4	99 000	0.88	− 0.12	0.0144
1	14.8	65 000	0.58	− 0.42	0.1764
2	20.5	108 000	0.96	− 0.04	0.0016
3	26.0	165 000	1.46	+ 0.46	0.2116

Average = 112 700 $\Sigma v^2 = 6.6912$ $h = 2.27$.

discussion, it being excessively cumbersome for useful computation. It appears to the writer, however, that the application of the theory to streams with long records shows that the formula of the author will often give far too high values to the flood to be expected in such a long period as 1 000 years—which is another term for saying ever expected. Thus, in the Connecticut River at Hartford, the author's formula **Mr. Pillsbury.**

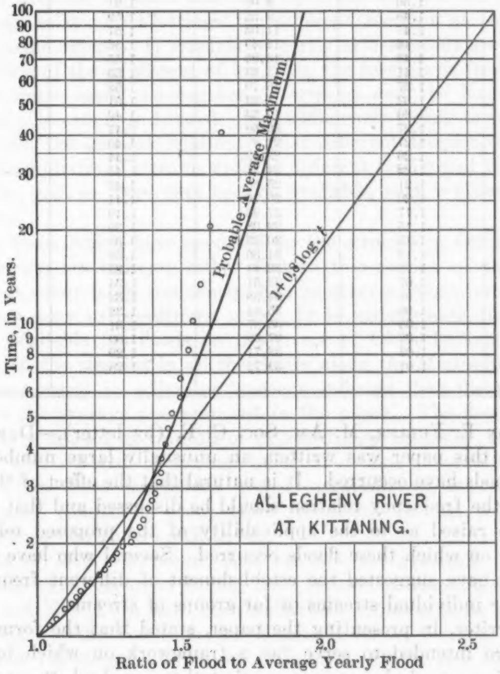

FIG. 16.

shows that a flood of 3.4 times the average, or of about 380 000 cu. ft. per sec., is to be expected in a period of 1 000 years. Dropping the somewhat cumbersome average maxima, the theory of probabilities indicates that the chance of a flood twice the average, or 225 000 cu. ft. per sec., is but one in 1 000. Would an engineer be justified in constructing works to care for more than, say, 250 000 cu. ft. per sec.?

Mr. Pillsbury.

TABLE 42.

No. of flood.	Ratio to average.	Summation.	Summation ÷ No. of flood.	Time, in years.
1	1.82	1.82	1.82	70.0
2	1.70	3.52	1.76	35.0
3	1.55	5.07	1.69	23.3
4	1.51	6.58	1.64	17.5
5	1.50	8.08	1.61	14.0
6	1.48	9.56	1.59	11.6
7	1.47	11.03	1.57	10.0
8	1.46	12.49	1.56	8.75
9	1.43	13.92	1.55	7.8
10	1.41	15.33	1.53	7.0
11	1.37	16.70	1.52	6.4
12	1.33	18.03	1.51	5.8
13	1.30	19.33	1.49	5.4
14	1.27	20.60	1.47	5.0
15	1.27	21.87	1.46	4.7
16	1.26	23.13	1.44	4.4
17	1.22	24.35	1.43	4.1
18	1.21	25.56	1.42	3.9
19	1.20	26.76	1.41	3.7
20	1.17	27.93	1.40	3.5
21	1.15	29.08	1.39	3.3
22	1.15	30.23	1.38	3.2
23	1.13	31.36	1.36	3.0
24	1.08	32.44	1.35	2.9
25	1.07	33.51	1.34	2.8
26	1.07	34.58	1.33	2.7
27	1.06	35.64	1.32	2.6
28	1.03	36.67	1.31	2.5
29	1.02	37.69	1.30	2.4
30	1.02	38.71	1.29	2.3
31	1.01	39.72	1.28	2.26

Mr. Fuller.

WESTON E. FULLER, M. AM. SOC. C. E. (by letter).—During 1913, and since this paper was written, an unusually large number of disastrous floods have occurred. It is natural that the effect of these great floods on the frequency relation should be discussed and that questions should be raised as to the applicability of the proposed relations to the rivers on which these floods occurred. Several who have discussed the paper have suggested the establishment of different frequency relations for individual streams or for groups of streams.

The writer, in presenting the paper, stated that the formulas proposed were intended to serve "as a framework on which to arrange the data in an orderly manner, so that they can be better understood and more readily used." With this object in view, formulas were derived which expressed the average frequency relation for the floods which have occurred on many rivers widely distributed over the country. Tables were presented giving values of the coefficient, C, as obtained from the average yearly flood for such rivers as had been observed a sufficient number of years to give an approximate idea of the size of this average. In these tables were also included other values of C, obtained by reducing the larger floods by the use of the proposed formula. These latter values of C indicate how the actual

floods which have occurred on the rivers have agreed with the proposed formula. The close agreement, for most of the rivers, of these two sets of coefficients indicates that the relation is a general one. During the course of the study on which the paper was based, plottings were made for many individual streams and for groups of streams in different sections of the country. The effect of the few larger floods on the plottings for individual streams was so great that the writer concluded that the relation indicated by them was less accurate than the average relation. Plottings for different sections of the country varied to some extent. If streams in partly arid sections are excluded, the variation of the coefficient of log. T in the formula is from 0.7 to 1. In the extreme cases the number of streams and the length of the records were too short to furnish proof sufficiently strong to justify any change from the general relation. That different frequency relations do exist for individual streams and for different sections of the country is probable, and, as more data become available, such relations may be established. Mr. Fuller.

Great floods which have occurred on the streams in Ohio since the paper was written indicate either that in this section of the country great floods occur more frequently than in other sections, or that these floods were very extraordinary ones. It is unfortunate that so few data are available for floods on the rivers of Ohio, Indiana, Illinois, and, in fact, the streams in all the States along the Mississippi Valley. Floods on streams in adjoining sections indicate that the relation is similar to the average one proposed in the paper. The floods of 1913 in Ohio were caused by rainfall of extraordinary intensity, considering the large area covered, occurring under conditions favorable for great floods. That these conditions were extraordinary is certain, but whether such conditions occur more frequently in this section than elsewhere only the future will show.

Floods of 1913.—Although data are not yet available for all the great floods which occurred during 1913, records of those on some rivers in Ohio and New York have been published. Among those of particular interest are the floods on the Miami River at Dayton, Ohio, the Scioto and Olentangy Rivers at Columbus, Ohio, and the Hudson River at Mechanicsville, N. Y. The data for these rivers, revised to include the records of 1913, are given in Table 43.

The value given for the ratio between the flood of 1913 and the average flood for the streams at Columbus corresponds to a value of T equal to about 1 400. As there are more than 1 400 records of floods on different rivers for which the ratios to the average flood are now available, the occasional occurrence of such a flood is not surprising.

For the Miami River at Dayton, Mr. Morgan expresses the opinion that the coefficient, C, in the writer's formula should not exceed 50. If this were the proper value for C, the 1913 flood on the Miami

Mr. Fuller.

TABLE 43.

Name of stream.	Location of station.	Catchment area, in square miles.	Flood Flows, in Cubic Feet per Second.		Period of observation, in years.	Ratio of maximum flood to average flood.	Probable value of C.
			Average yearly.	Floods of 1913.			
Upper Scioto...	Water-works dam...	1 032	19 300	68 000	16	3.52	75
Olentangy......	Columbus, Ohio.....	520	14 500	51 000	16	3.52	97
Lower Scioto...	Columbus, Ohio.....	1 570	33 800	119 000	16	3.52	94
Miami..........	Dayton, Ohio.......	2 450	50 000*	246 000*	21	4.92	90
Hudson.........	Mechanicsville, N. Y.	4 500	44 500	108 000	23	2.42	53

* Represents rate at maximum stage.

The data for the floods at Columbus are from the "Report on Flood Protection for the City of Columbus" by John W. Alvord and C. B. Burdick, Members, Am. Soc. C. E.

The data for the maximum flood at Dayton are from the discussion by Arthur E. Morgan, M. Am. Soc. C. E.

The data for the maximum flood on the Hudson are from *Engineering Record*, April 12th, 1913, "Effect of Recent Floods on New York Streams" by R. E. Horton, M. Am. Soc. C. E.

River would be relatively very greatly in excess of any flood which we have known on other rivers. The data on which to base the value of C for the Miami are not very satisfactory. In the papers of the U. S. Geological Survey there are 4 years, 1906 to 1909, inclusive, for which floods at Dayton are recorded. An average of these four floods would indicate a value of C of about 86. The best indication of the average flood at Dayton is that deduced from the gauge heights of the U. S. Weather Bureau, which are available for 21 years. As the rating curve for large floods is indefinite, it seems best to obtain the average flood by means of the median gauge height during this period. This has been 11.9, corresponding to a flood of about 39 000 cu. ft. per sec., according to the rating by the U. S. Geological Survey in 1906. During the last 10 years the median gauge height has been 13.25, which corresponds to a flood of about 45 000 cu. ft. per sec. From other rivers it has been found that the median flood is usually less than the average by about 10 per cent. On this basis the average yearly flood on the Miami at Dayton, on the 24-hour basis, under present conditions, is probably at least 45 000, and the value of C is at least 90. The corresponding maximum rate of flow would be 50 000 cu. ft. per sec., or more.

During the past 20 years many changes have been made in the channel of the Miami, such as encroachment on the river channel in cities, and the construction of bridges with long approaches of solid embankment, which greatly reduce the waterway. It seems probable that the coefficient of C for the Miami River at the present time is different from what it was a number of years ago. Mr. Morgan thinks that the Miami should have a comparatively low coefficient of flow on account of the comparatively small average slope of the water-shed. Steep slopes on the upper branches with smaller slopes on the main

Mr. Fuller.

river and lower branches are conditions which may produce larger floods than uniform steep slopes. A comparison of the coefficients for different rivers of the same general character fails to indicate any great effect from the average slope of a river. The writer suggests that the relation between the slopes on different parts of the river and on its branches is of greater importance in determining the flood-producing capacity than the average slope.

The maximum flood on the Miami in 1913 may have been very different in magnitude from the flood which would have occurred if the channel had been unobstructed. The large number of embankments thrown across the valley, with inadequate waterways, provided temporary storage which tended to retard the earlier run-off until the branch streams poured in their maximum flood flows. The subsequent failure of these embankments releasing the stored waters, probably increased the flood materially.

Taking these matters into consideration, together with the great uncertainty of gauging such a flood, it seems probable that the recorded flood flow may have been considerably in excess of the flood which would have come under natural conditions. On the whole, the writer believes that the Miami flood of 1913 was not greater than five times the average yearly flood under present conditions, and that it may have been much less. At all events, the flood was a very exceptional one, and must be given careful consideration in determining the probable floods on our rivers, particularly those in the central part of the country.

The flood on the Hudson River in 1913 is an interesting one, from the standpoint of what may happen on other rivers. The maximum flood on the Hudson, for which records for many years are available, was, previous to this year, much less than what would normally be expected on the basis of its average flood. The flood of 1913, however, not only reached the normal maximum flood for the period of record, but exceeded it by a considerable quantity. There are many other rivers on which the maximum recorded flood is much less than the flood indicated by the average relation. That much greater floods will occur on some of these rivers within a comparatively short period seems assured. As an instance, the greatest flood on record on the Ohio at Wheeling and at Pittsburgh is equivalent to a flood which may normally be expected in a period of not more than 15 years. If a flood of the relative size of that on the Hudson occurs on the Ohio, it will mean a flood of more than 600 000 cu. ft. per sec. at Pittsburgh, or a flood from 4 to 5 ft. higher than the highest now on record.

Reliability of Frequency Relation as Established from Records on One Stream.—Messrs. Knowles, Horton, and Pillsbury give frequency relations for individual streams, which differ from the average relation

Mr. Fuller.

proposed by the writer. To show how unreliable such relations may be, Figs. 17 and 18 are presented, which show how greatly one or two of the larger floods affect the relation, even when derived from the longest records available for any stream.

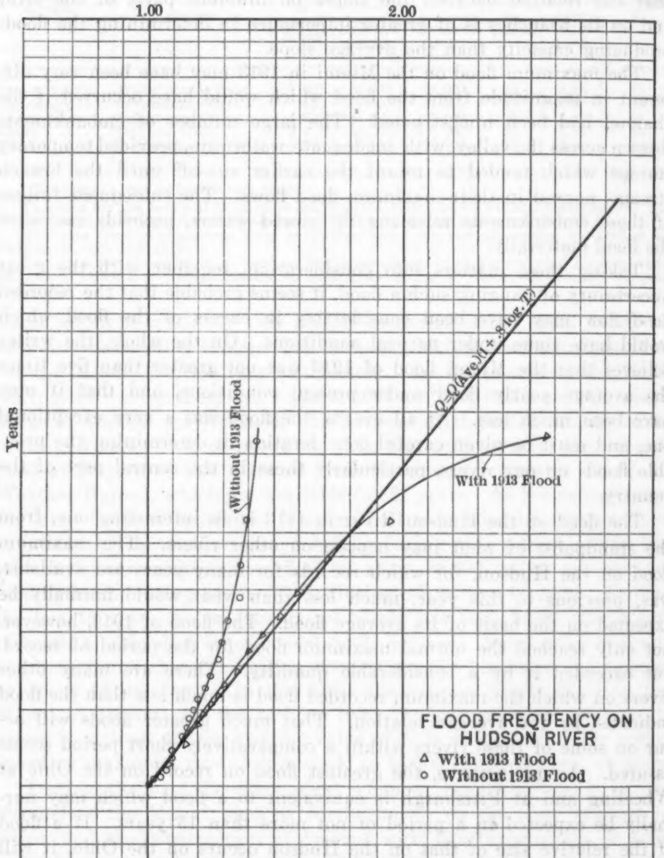

FIG. 17.

On Fig. 17 two curves illustrate the effect of a single great flood. The flood of 1913 on the Hudson is the greatest of which we have record. The effect of this flood is sufficient to change the data so that

the indications are just the reverse of what they were before the flood occurred. In other words, prior to 1913, the Hudson River records indicated that great floods occurred less frequently than on the average river. By including this flood in the data, the indications are that such floods occur with a greater frequency. Mr. Fuller.

Mr. Pillsbury contributes data for floods on the Connecticut River for 70 years, and Fig. 15 shows a frequency relation based on these data. Fig. 18 shows what the effect would be if two of the floods (which were actually about 1.5 times the average) had been somewhat greater. With this slight change the curves would be practically identical with the curves for eastern rivers, as shown on Fig. 1. In other words, the data for the Connecticut indicate a difference in frequency relation from the average of all eastern rivers only in that two floods are smaller than the normal. If any two of the seven or eight largest floods had happened to be greater by from 30 to 40%, the relation would have agreed with the general one. That such floods will occur on the Connecticut, and that, in another 70-year period, the data may indicate a relation equal to or exceeding the normal one, is probable. There are no continuous records for any streams in America of a length greater than that on the Connecticut, and the writer submits that curves, based on single records like those given in the discussions, do not justify any change of the relation from the average, unless other evidence is produced.

Comparison of Proposed Formula with Others.—Mr. Kuichling suggests a new formula, for use in the South Atlantic States, similar in form to his other well-known formulas. This and other formulas derived in a similar way give relatively higher values for small streams than the writer's formula. There is an essential difference in the meaning of these formulas which should be understood. The writer's formula gives the flood which will probably occur on the particular stream in question in a given interval of time. Mr. Kuichling's and other formulas, derived by plotting the maximum floods which have occurred on streams of different sizes, give the greatest flood which has occurred on any of the large number of streams in the varied intervals covered by the several records.

As there are many more small streams than large ones, it is obvious that there are more chances of obtaining an extraordinary flood on some one of the many small streams than there are of obtaining a similar flood on one of the few large ones.

As an illustration, take Mr. Kuichling's formula for the rivers of the South Atlantic States, which he states is

"based on the greatest observed discharges of the Potomac River at Point of Rocks, Md., the New River at Radford, Va., the Catawba River at Rock Hill, N. C., the Little Tennessee River at Judson, N. C., Cane Creek at Bakersville, N. C., and numerous other streams which

Mr. Fuller. exhibit somewhat smaller rates of discharge than the preceding. This new formula is

$$q_{max.} = \frac{41.6\,(620 + M)}{24 + M} \qquad (3)$$

in cubic feet per second per square mile, and it may be regarded as applicable to mountainous and hilly water-sheds having areas of not more than 10 000 sq. miles, in the portion of the country indicated."

There are probably fully 1 000 streams similar in size to that of Cane Creek for each one of the size of the Potomac. It is then to be expected that there will be many times as many chances of obtaining a single great flood on some one stream like Cane Creek as there will be of obtaining a flood relatively as great on a stream like the Potomac.

The formulas thus derived have an unbalanced element which must be taken into account. Mr. Kuichling has allowed for this to some extent, as his formula gives values much less than the recorded flood on Cane Creek, but the writer believes that the value given by the formula is still relatively too high.

The recorded sizes of floods, on streams like Cane Creek, Devil's Creek, and others which are greatly in excess of any well-verified flood discharges, are of doubtful value. A study of the methods used in gauging such floods shows that the measurements are obtained from slopes and sections taken after the flood. Little is known of the conditions which existed during the flood, and the apparent slope is often in error. High-water marks may have been caused by the backing up of the water by obstructions which afterward passed on under the flood pressures. The failure of structures may have caused great discharges for a short interval, which would not have occurred if the obstructions had not existed. The effect on the rate of flow of a stream due to the carrying of great quantities of débris which catch on fences, trees, and other obstacles causing eddies and reducing the channel area, is not known. If one may judge from experience with obstructions in pipes, this effect must be large. The effect of washing away the banks, the failure of bridges and dams, and of other matters which occur during such disastrous floods, is but little known. Flood discharges obtained by such methods are often not even approximately correct, and too much dependence should not be placed on them.

It may be well to state here that, in comparing the writer's formulas with others, the formula, $Q\ (\text{max.}) = C\ A^{0.8}\ (1 + 0.8 \log. T) \left(1 + \frac{2}{A^{0.3}}\right)$, should be used, as in most cases at least the maximum rate of flow is given by the other formulas.

Selecting a Value for C.—The selection of the proper value of C is important in the use of the proposed formula. Before its selection, a study of the flood data on the stream in question and on other

Mr. Fuller.

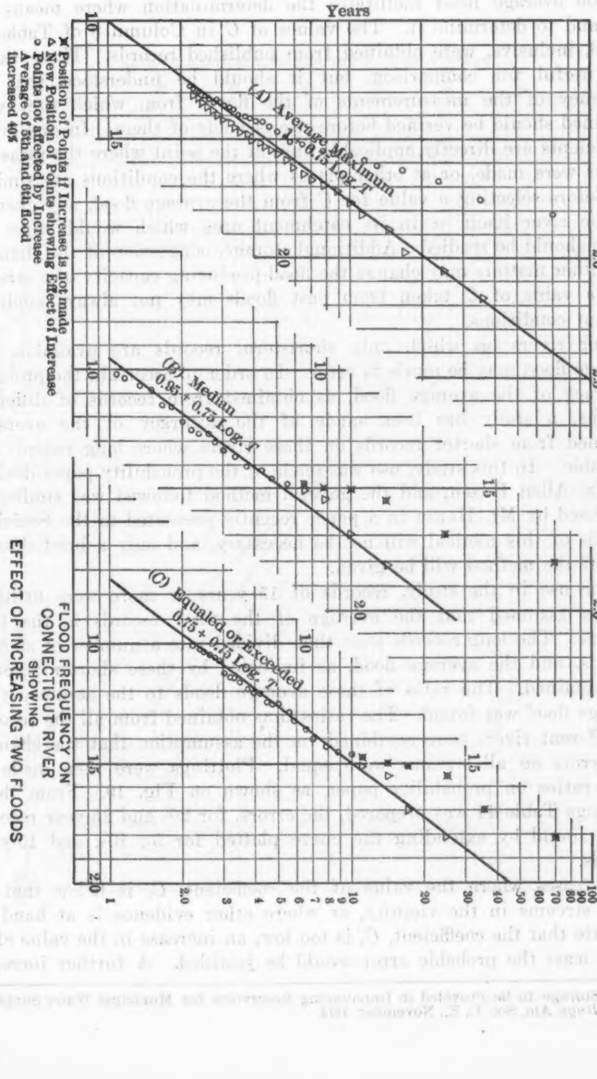

FIG. 18.

FLOOD FREQUENCY ON CONNECTICUT RIVER SHOWING EFFECT OF INCREASING TWO FLOODS

Mr. Fuller. streams of similar nature in the vicinity should be made. This study should be just as complete and thorough as it would be if any other method of determining the maximum flood were to be used. The use of the average flood facilitates the determination where means are at hand to determine it. The values of *C* in Column 8 of Tables 12 to 26, inclusive, were obtained from published records. These values are useful for comparison, but it should be understood that the accuracy of the measurements of the floods from which they were obtained should be verified before use is made of them. In any event, coefficients are directly applicable only at the point where the measurements were made, or at other places where the conditions are similar.

Before selecting a value for *C* from the average flood, any changes in the river itself or in its catchment area which would affect the value should be studied. Additional storage, congestion of the channel, and other matters may change the flood-producing capacity of a stream, and a value of *C* taken from past floods may not always apply to present conditions.

For rivers on which only short-term records are available, the average flood may be much in error. In order to ascertain the probable accuracy of the average flood, as obtained from records of different lengths, a study has been made of the accuracy of the averages obtained from shorter records on those rivers where long records are available. In this study, use was made of the probability paper devised by Mr. Allen Hazen, and the general method followed was similar to that used by Mr. Hazen in a paper recently presented to the Society.* Details of this method will not be necessary, and only a brief discussion of the method will be given.

For use in the study, records of 15 years or more were utilized. It was assumed that the average of the total records is the true average. The long records were then divided into a number of shorter records, and the average flood, as indicated by these shorter records, was obtained. The ratio of these average floods to the assumed true average flood was found. The ratios thus obtained from all the records of different rivers were combined, on the assumption that the chances for errors on all streams were equal. Plottings were then made of these ratios on probability paper, as shown on Fig. 19. From these plottings Table 44 was prepared, the errors, for 20- and 25-year records being found by extending the curve plotted for 5-, 10-, and 15-year records.

In cases where the value of the coefficient, *C*, is below that of other streams in the vicinity, or where other evidence is at hand to indicate that the coefficient, *C*, is too low, an increase in the value of *C* by at least the probable error would be justified. A further increase

* "Storage to be Provided in Impounding Reservoirs for Municipal Water Supply," *Proceedings*, Am. Soc. C. E., November, 1913.

Mr. Fuller.

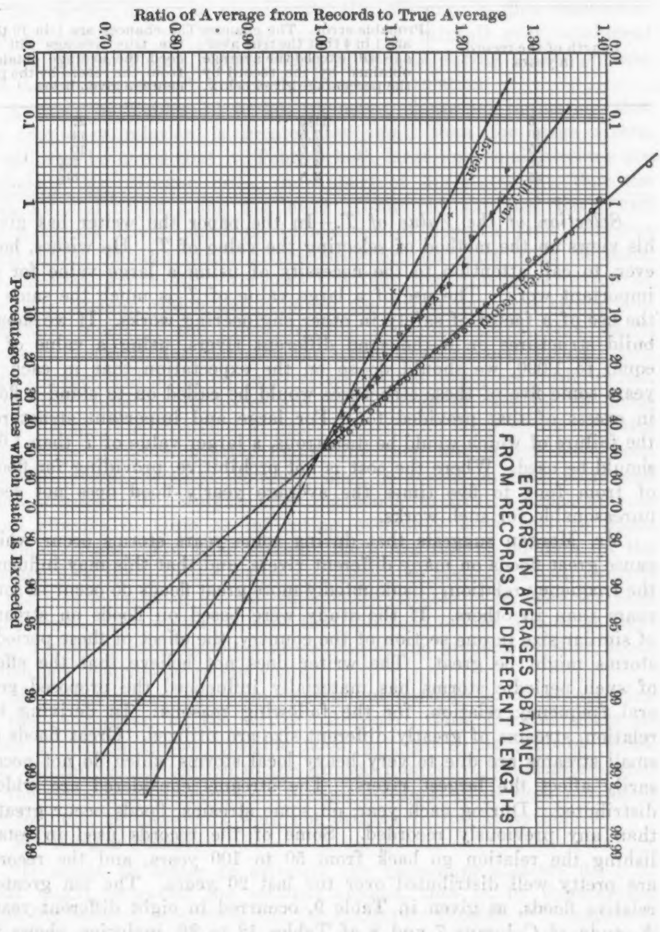

FIG. 19.

Mr. Fuller.

to provide a factor of safety depends largely on the importance of the works under consideration, and on how great a factor of safety has otherwise been provided for in the selection of the value of *T*.

TABLE 44.

Length of the record, in years.	Probable error. The chances are 1 in 4 that the true average will exceed the average obtained by the record by the percentage given below.	The chances are 1 in 10 that the true average will exceed the average obtained from the record by the percentage given below.
5	11½	22
10	7.5	14
15	5.5	10
20	4.0	7½
25	3.5	6½

Selection of the Value of T.—In the paper the writer has given his views on the method of selecting the value of *T*. He wishes, however, to call attention to the necessity of using a large value for all important work. The use of a large value of *T* is much the same as the use of a factor of safety in other engineering works. If we should build structures on a thousand different rivers, using a value of *T* equal to 1 000, we should do so on the expectation that in each 10 years some five of these structures would be called on to stand a flood in excess of that provided for. For large and important structures, the failure of which would be disastrous, a larger value of *T* than 1 000 should be used. Where the cost is not prohibitive, providing for floods of from four to five times the average yearly flood does not seem unreasonable for such works.

Mr. Morgan suggests that during some years storms occur which cause great floods on many different rivers, and that this may influence the frequency relation. Undoubtedly more great floods do occur in some years than in others. If the study were based on floods on streams of similar size in one section of the country, the effect of these periodic storms might be great. The writer does not believe that the effect of such periodic storms has materially influenced the proposed general frequency relation, for the following reasons: In deriving the relation, streams of greatly different size are utilized. Great floods on small streams are due to very heavy local storms which do not necessarily affect the largest rivers. The streams considered are widely distributed. During each year, on some streams, floods occur greater than any previously recorded. Some of the records used in establishing the relation go back from 50 to 100 years, and the records are pretty well distributed over the last 20 years. The ten greatest relative floods, as given in Table 9, occurred in eight different years. A study of Columns 7 and 8 of Tables 12 to 26, inclusive, shows no difference in frequency between streams with long and with short records.

Mr. Morgan states: Mr. Fuller.

"On large water-sheds of 10 000 sq. miles or more, excessive rainfall in one part of the water-shed is usually balanced by lack of rainfall in another part, and the ratio between the average annual flood and the maximum possible flood must be less than for small areas."

The writer does not agree with Mr. Morgan in this statement. The difference in the average intensity of rainfall over catchment areas of different sizes occurs yearly as well as during longer periods, and does not necessarily affect the frequency relation.

A study of Tables 12 to 26, inclusive, shows no indication that the frequency relation is greater for small than for large rivers. It will also be noted that in Table 9 both large and small streams are included. Mr. Morgan cites, in support of his statement, that the large alluvial rivers along the lower Mississippi, the Red River and the Arkansas River, show no indications of deposits from previous great floods, though small rivers elsewhere do show such deposits. Mr. Morgan apparently overlooks the fact that the construction of the levee system, which has confined the floods on these rivers, and prevented much of the great overflow or temporary storage which formerly occurred, has increased the stages during recent floods. Still higher stages will occur on these rivers, as the levees are built higher, until such times as the levees are high enough to care for all floods which occur. The existence of gravel deposits on other rivers indicates past floods of much greater magnitude than any which have been recorded. Such deposits, however, may be found on both large and small streams. Mr. Morgan cites the floods on Devil's Creek and on small streams in Ohio as indicating extremely large relative floods on small rivers. For such streams there are few or no data on the average flood, and, in the writer's opinion, the recorded measurements of the maximum flood are not sufficiently reliable to be used for such comparison. The writer believes that the flood on the Miami at Dayton—a river of considerable size—is one of the greatest relative floods which has occurred in recent years.

Mr. Morgan uses the term "maximum possible flood" as applying to the flood given by the writer's formula. This formula gives no limit to the maximum possible flood, but gives the probable flood. Mr. Morgan calls attention to the limited number of years represented in the table of great floods on foreign streams. This does not affect the formula, as these data were not utilized in its derivation.

Mr. Morgan states:

"It might be better to estimate the maximum possible flood by what might be called the rational method, that is, by determining from the basis of experience and the maximum rainfall to be expected, the relation of rainfall to run-off under the conditions which would exist in an assumed case, considering the elements of topography, shape of the drainage basin, direction of storms, season of the year, etc."

Mr. Fuller.

There is quite as much uncertainty in probable maximum rainfall as in probable flood flow. There is, in addition, great uncertainty as to the percentage of run-off to be expected on different rivers, and under different conditions. We know but little of the effect of topography, direction of storm, shape of catchment area, and other factors. To the writer there seems to be much less chance for error in selecting the value of C from the average yearly flood, or by a study of the values of C for other rivers, than in estimating values for these many unknown quantities.

Many of the important points brought out in Mr. Morgan's interesting and instructive contribution have been covered in the writer's general discussion, and need no further comment.

Mr. Hinckley's interesting remarks, made in 1911, indicate clearly how engineers have looked at the probability of the occurrence of floods. His rainfall data are interesting. The application of probability methods for obtaining the probable rainfall would undoubtedly give much valuable information.

Mr. Chandler presents some valuable data for the flow of the Red River of the North and its branches, and draws some interesting conclusions as to the frequency relation of streams in that section. He says that the conditions in the Red River Valley are similar to those in a partly arid region, so that the writer's average relation does not hold. Although the data available are not sufficient to be conclusive, Mr. Chandler's suggested frequency relation gives an indication of how much higher it may be in such sections. The size of the average flood for such rivers is governed largely by the number of dry years in the period. It occurs to the writer that in determining the average flood, the exclusion from the record of all years so dry that no real flood occurs, may have merits, thus giving an average of the real floods as a basis of comparison. On this basis the relation may be expected to be more like that for other sections.

The writer notes that Mr. Chandler very properly uses the median flood method in his study. This method is better for plotting short records, as the few largest floods do not affect the plotting of the other floods to the same extent as in the average maximum flood method, and by its use a closer approximation to the true curve is obtained.

Mr. Hazen points out clearly the uses and limitations of the proposed formulas. The map of the country (Fig. 8) is most useful, as it indicates the data available for different sections of the country and also shows the variation in the value of C for rivers in the same section. A study of the conditions will in most cases account for such variations. For instance, in Maine, where the coefficients vary from 17 to 110, it will be found that the low coefficients are for small streams draining extensive systems of lakes which control largely the floods, and the large values are for streams on which there is little or no storage.

It is interesting to study the effect of this storage on the river system. For the upper branches, the effect is great, but, as the streams join to form the larger rivers, the effect decreases rapidly until, near the mouth of the river, the coefficients are in some cases several times as great as for the upper branches. The study of streams in that State, where there are such extensive lake systems, is most illuminating in regard to storage for flood protection. Mr. Fuller.

As Mr. Hazen states, probability paper might have been utilized in the study of flood frequency. During the study for this paper the normal probability curve was tried, but it was found that the data did not fit it as closely as the logarithmic curve adopted. The use of probability paper, which provides a ready means of drawing curves varying somewhat from the normal law, allows this method to be used. Practically, however, such a curve would give results identical with the formula, within the limits of the data, and an extension of the curve would indicate a ratio of 4.0 for $T = 10\,000$, as compared with 4.2 by the formula. Such differences are of little moment, and no data are available to indicate which is the more nearly correct.

The method of plotting gauge heights on probability paper, as suggested by Mr. Hazen, should prove useful. The writer has plotted records of the gauge heights for many stations along the Mississippi and its branches. Although these plottings are not in all cases as close to a straight line as the one for Cincinnati, they all approximate such a line. Within such limitations as may be applied by one having a thorough knowledge of the river, both as to changed conditions affecting the stages and as to the storage and increased channel capacity which occur at higher stages, this method seems applicable to many rivers.

Mr. Hazen's suggestion in regard to determining coefficients of variation, as an index of how closely the stream follows the average law of frequency, is interesting. It may be that a thorough study of the regularity of flow throughout the year, and of other characteristics of the stream, would enable the effect of some factors on flood frequency to be ascertained.

Mr. Knowles, citing a record of a single stream as an indication of variation from the average, warns against the general use of the average relation. The writer pointed out in the paper, and has further stated in this discussion, that a thorough study of all local conditions should be made before using the formula, but must repeat that, in his opinion, the data on a single stream are entirely inadequate for establishing a frequency relation. It could be shown by a plotting similar to Fig. 18 that the data for the Allegheny River at Kittanning really differ but slightly from the average relation. Mr. Knowles' extension of the existing record to a 41-year record, by estimating the flood at Kittanning from the gauge height at other points on the river

Mr. Fuller. is of doubtful accuracy. An examination of Figs. 11 and 12 will show that the actual floods which Mr. Knowles plotted in order to obtain the curves showing the relation between the gauge heights at Kittanning and those at Freeport and Parker differ, in some instances, by from 50 to 100% from the curves. If these actual floods do not agree more closely with the curves, it is obvious that similar errors probably exist in the floods obtained in this way and included in the record. As the effect of the few large floods is so important, it is clear that a record obtained by such methods should not be used in discussing frequency.

Mr. Bellamy's discussion is interesting, and gives much valuable information as to flood flows in Australia. The large variation in floods for the different rivers is to be expected in a country so large and with such widely varying conditions. As Mr. Bellamy states, a comparison of such rivers, with the limited data available, is very unsatisfactory. It is much the same as comparing rivers of the American coast, where the coefficients are in many cases greater than 100, with rivers in the Missouri River Basin or the Great Basin, where the coefficients are in many cases less than 10.

Mr. Kuichling has supplied tabulated data relating to great floods which have occurred on both foreign and American rivers, and these, taken in connection with those previously presented by him, give by far the most complete record of maximum flood flows available. The data from which this information was prepared are widely scattered. In many cases, particularly for foreign streams, the original data are in such a state that most careful study of them is required before they can be used. The collection and analysis of this large mass of data should be appreciated by all who are interested in the flood problem.

Mr. Kuichling brings out clearly many important aspects in regard to flood flows. Some of these points have been taken up by the writer in the preceding general discussion, and need no further comment.

Mr. Kuichling calls attention to the wide variation in the value of C for different rivers in the same general section of the country, and suggests that it may be better to use a single value of C for one section. Where coefficients are based on long records, the writer believes that the differences in the values of C indicate different flood-producing capacities for the streams. In many cases these differences may be accounted for by a study of the catchment area, as to slopes, storage capacity in lakes, and rainfall conditions. Where records of considerable length are not available, the writer agrees with Mr. Kuichling that much consideration should be given to the coefficients for other streams in the section. He is correct, in his interpretation of Tables 12 to 26, inclusive, in stating that the values of C in Column 8 were the ones intended for general use. The values in Column 8 are derived from the average flood, and take into account all the largest yearly floods in the rivers; they are more accurate than those in Col-

umn 7. It would have been better, as he suggests, to have called these C_1 and C_2. Mr. Fuller.

Mr. Kuichling calls attention to the important work of Iszkowski, and reduces his formula to conditions for the United States. Although this formula is of very different form from the writer's, it may be of interest to show how the results obtained from it agree with those obtained from the writer's formula,

$$Q \text{ (max.)} = C\, A^{0.8} (1 + 0.8 \log. T)\left(1 + \frac{2}{A^{0.3}}\right).$$

Under what may be regarded as similar conditions, Iszkowski's formula differs from the writer's by the following percentages: for a catchment area of 10 sq. miles, — 21%; 100 sq. miles, + 32%; 1 000 sq. miles, — 2%; 10 000 sq. miles, — 39 per cent. This comparison is made on the basis of the values deduced by Mr. Kuichling from Iszkowski's formula for "hilly country, slightly permeable soil, and sparse vegetation" and of a use of $C = 100$ and $T = 100$ in the writer's formula.

The writer has plotted the largest of the floods in America, as given by Mr. Kuichling, on the lower diagram of Plate XI, and finds that with the exception of three or four extreme floods, such as those on Cane Creek and Devil's Creek, the plottings agree with the curves representing the writer's formula as well as the points previously plotted.

Mr. Horton calls attention to a discussion of the use of probability methods in a report made by Mr. Rafter in 1896. This discussion is interesting and instructive. That paper, however, is entirely confined to a discussion of rainfall and minimum run-off. The writer fails to find any suggestion as to the use of similar methods for determining flood flows. Until Mr. Horton called his attention to it, the writer was not aware of the existence of this study. Mr. Horton further states that Mr. Rafter suggested to him the use of probability methods for determining the probable flood as early as 1896 and that he has used it on numerous occasions in his professional work. The writer had no means of knowing about any work which Mr. Horton had done on this subject, and sees no reason why he should qualify the statement that the original suggestion came to him from Mr. Hazen.

Mr. Horton gives three different formulas derived from the data on rivers near Philadelphia. He also gives the following general formula for which he states that he

"does not claim any great breadth of applicability for the general formula, * * * but believes that factors other than area modify the flood discharge of streams so profoundly that it is better, wherever possible, to derive individual formulas or flood-frequency diagrams for each stream."

Mr. Fuller.

This formula is:

$$Q = 4\,021.5 \frac{T^{\frac{1}{4}}}{A},$$

in which Q is the flow in cubic feet per second per square mile. To put this formula on the basis of the writer's, that is, to give the total flood flow of the stream, it becomes necessary to multiply by A, so that the formula becomes

$$Q = 4\,021.5\ T^{\frac{1}{4}}.$$

According to this formula, the maximum floods to be expected are independent of the catchment area. Surely a formula which does not take into account the size of the catchment area can have no general application.

During the early part of the study for this paper, exponential formulas such as those proposed by Mr. Horton were tried. It was found that though these gave curves closely following the data for short periods, for the longer records the exponential relation was less satisfactory than the logarithmic relation adopted.

The writer deems it unfortunate that, after seventeen or more years of consideration of the use of probability methods for the determination of flood flows, Mr. Horton should have confined his discussion so largely to criticizing details of the writer's methods, instead of giving more in regard to his own studies.

Mr. Horton states:

"It would appear that the relation between flood magnitude and frequency, when expressed in terms of either 'average maximum flood' or 'median flood,' is not only very indefinite and difficult of comprehension, but is apt to be very misleading, if it is not indeed practically meaningless. It does not directly convey the information which the engineer usually most desires, for example: If a spillway has a capacity of 1 000 cu. ft. per sec., how often on an average will its capacity be exceeded?"

In regard to this statement, the writer will call attention to the fact that the others who have discussed the paper have not found the proposed formula either indefinite or misleading, but, on the contrary, have comprehended its true meaning. The writer, on pages 575-582, described three different methods which are applicable to flood frequency problems. In Table 6 the formulas representing these methods are stated, and values are given for the volume of the flood indicated by them in different periods of time. If any one wishes to know the volume of the flood that will probably be equalled or exceeded in a period of time, he may do so by using the formula:

$$Q \text{ (equalled or exceeded)} = C\,A^{0.8}\,(0.7 + 0.8 \log. T)$$

as given in Table 6. There are problems in which it is useful to know the volume of this flood, but the writer believes that the most important question which the engineer desires answered is: What is the probable flood for which to design our structures? For this purpose we surely do not wish to ascertain the smallest of the maximum floods that is likely to occur in the given interval, which is only a different way of stating the "flood to be equalled or exceeded." For example, suppose we have a record which includes ten periods, each of 20 years. Each of these periods would have a maximum flood of a different size. The flood to be equalled or exceeded in a 20-year period would be the smallest of these. To design works which would just provide for this flood would mean that in all probability the design would prove inadequate within the period, because during nine out of ten periods, greater floods would occur. Mr. Fuller.

The average of the ten maximum floods which have occurred in the several periods, as given by the writer's formula, seems to be a logical basis for design in order to provide for a reasonable chance that the structure will survive the given interval. The median of these floods, or the one for which there is one greater for each one less, represents the flood for which the chances are even that it will occur in the given period. The use of this flood is as logical as the use of the average. The difference in value between the two is slight, and the writer, for the reasons stated in the paper, prefers the average maximum.

Mr. Horton, after giving a method of determining the "average interval of recurrence of a flood lying between any two given magnitudes", states: "This illustrates what seems to be a fatal error in the author's method of analysis."

The writer fails to find in the paper any statement which would justify the interpretation Mr. Horton has given the formula. As the method of derivation and the meaning of the formulas are explained in considerable detail in the paper, the writer considers that to apply it as Mr. Horton has done is entirely unwarranted. If the determination of the average interval of recurrence of floods of a size between fixed limits is desirable, as it may be for some special problem, the use of the flood to be equalled or exceeded, as suggested by Mr. Horton, is proper. Such use, however, is an unusual one, and to find a "fatal error" in the average maximum flood formula because it is not applicable to such a problem, is like condemning a formula proposed for determining the strength of a steel beam because it does not give the deflection of the beam. The writer was aware that the "flood to be equalled or exceeded" and the "median flood" had their uses, and accordingly gave the formula by which they might be obtained and descriptions of the methods of derivation, but, to

Mr. Fuller. explain their application to special problems, he thinks is beyond the scope of the paper.

Mr. Horton considers the use of ratios "confusing and cumbersome". By the use of ratios it becomes possible to compare the frequency of occurrence of floods on different rivers, which, in the writer's opinion, is of the greatest importance.

Mr. Horton objects to the writer's method in that he does not use all the floods to determine the frequency relation. As a matter of fact, in the determination of the general formula, 1 672 floods were considered; 20 of the largest were plotted individually and 200 others in groups of ten. This plotting covered three-quarters of the total length of the curve, as plotted in Fig. 1. It is apparent that, with the average maximum method, the average flood, which has a ratio of unity, must be plotted for 1 year. A straight line drawn from unity through the average of the points plotted satisfied the requirements, thus making the plotting of the remaining points so obviously unnecessary that it was not done. The writer assures Mr. Horton that the remaining points will fall close to the line. Mr. Horton's other criticisms of the paper are, the writer believes, fully covered by the foregoing discussion.

Mr. Pillsbury's discussion is an interesting and natural one. An examination of Fig. 18 shows how a slight change in the data on the Connecticut River would affect the frequency relation, and the writer has already given his views on the reliability of frequency relations established from the records of single streams. Aside from this, the assumption made by Mr. Pillsbury contains a fundamental error which would have to be, and could be, eliminated. This error is in assuming that the series of annual floods follows the normal law of probability. By the normal law of probability, the variations upward and downward from the normal are equal. As a matter of fact, with flood data, the variations upward and downward are not equal. The variations downward are more numerous, but the variations upward are greater in magnitude. The values follow, not the curve of normal probabilities, but what is called a skew curve. If Mr. Pillsbury will deduce the formula that most accurately represents this skew curve, and will show its application, he will give a useful solution of the problem which he has attempted, and will be doing the Profession a real service.

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Paper No. 1294

CONCRETE BRIDGES: SOME IMPORTANT FEATURES IN THEIR DESIGN.*

BY WALTER M. SMITH, SR., M. AM. SOC. C. E., AND WALTER M. SMITH, JR., JUN. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. S. W. BOWEN, WILSON FITCH SMITH, C. E. GREGORY, HENRY H. QUIMBY, A. C. JANNI, PHILIP AYLETT, L. J. MENSCH, A. W. BUEL, W. D. MAXWELL, AND WALTER M. SMITH, SR.

It is the object of this paper to bring out several important features in some concrete bridges recently designed by the senior writer.

Reinforced concrete is now being used in the construction of bridges of all three types: arch, girder, and truss; in fact, there is a bridge across the Seine at Avranches, France, which combines all three types: an arch span of 110 ft., two girder spans of 34 ft. each, and a truss span of 100 ft. There are two girder bridges near Pittsburgh, Pa., one of 75, and the other of 67 ft. span. In Nashville, Tenn., there is a railroad bridge which has two 95-ft. spans, consisting of reinforced concrete bowstring trusses.

The great advantage of concrete over stone for bridges is, of course, in its economy. The concrete bridge has an additional advantage in being stronger and much more reliable, due to the absence of joints.

The mortar in the joints of a stone arch is only from one-fifth to one-tenth as strong as the stone, and all the joints are never completely

* Presented at the meeting of November 5th, 1912.

filled with mortar. In arch work it is very difficult to fill completely with mortar an accurately cut joint of considerable area and small thickness. The writers are very strongly of the opinion that there has never been constructed a large stone arch in which every joint was completely filled with mortar. They have seen first-class stone and brick masonry torn down, in which not more than 75% of the space in the joints was filled with mortar. On the other hand, they have often had to excavate concrete (which had not been built with special care, but with a very cheap class of labor) in which there were no voids whatever.

The following is an example of the difference in cost of construction of concrete and stone arches. In the Manhattan anchorage of the Manhattan Bridge in New York City, the cost of the labor alone in setting the stone in arch work was $5 per cu. yd. The stone itself cost about $25 per cu. yd., delivered at the site, therefore the cost of the finished masonry, including cement, was more than $30 per cu. yd. The cost of this arch stone would have been much greater had it not been for the fact that it was included in a contract for a very large quantity of coursed dimension stone masonry, so that the cost of the latter governed the price of the arch work.

In the Rye Outlet Bridge, a reinforced concrete arch bridge of five spans of about 127 ft. each, designed by the senior writer, and built by the New York Board of Water Supply, near Valhalla, N. Y., each arch span, containing about 400 cu. yd., was built in an average of 10 hours. There were about 40 men working 10 hours on each span, therefore there were 400 man-hours on each span, or an average of 1 cu. yd. per man per hour. These men were paid from $1.50 to $1.75 per day of 8 hours. The cost of the labor, therefore, was about $0.22 per cu. yd. The net cost of the materials was about $4.25 for concrete, and $5 for steel, forms, etc., giving a total net cost of about $10 per cu. yd., as compared with a little more than $30 for stone masonry. The actual price of this concrete, including the contractor's profit, was $12.50, and of the stone about $34.00.

There is one case in which the stone bridge is preferable to the concrete. If esthetic features are paramount, and economy is not to be considered, then the stone bridge, on account of its adaptability to architectural ornamentation may be more desirable; yet, even in this

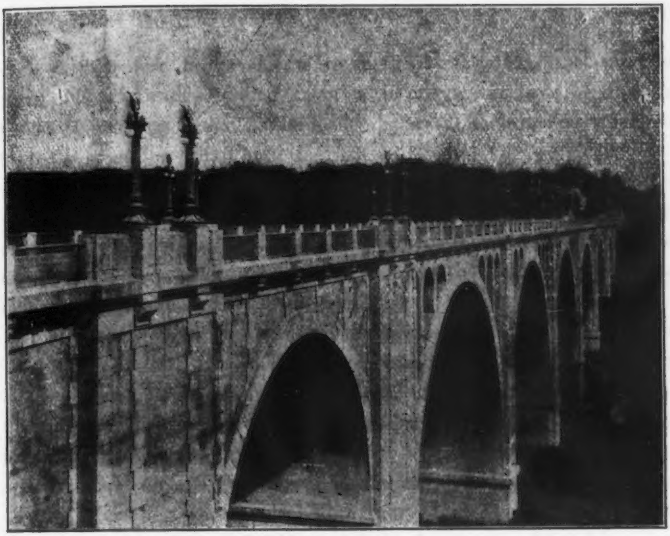

FIG. 1.—CONNECTICUT AVENUE CONCRETE ARCH BRIDGE, WASHINGTON, D. C.

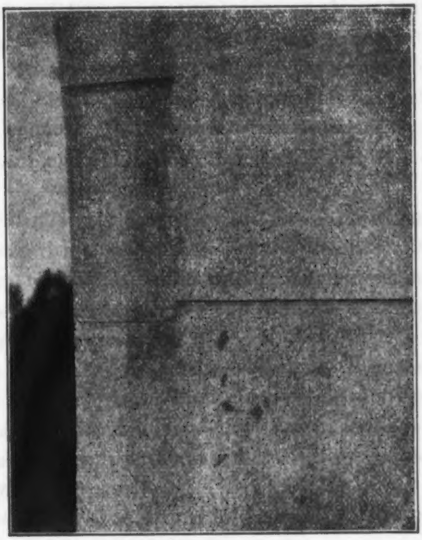

FIG. 2.—TOOL-DRESSED FACES OF BLOCKS, CONNECTICUT AVENUE BRIDGE.

[illegible]

Fig. 2.—Tool-Dressed Faces of Blocks,
[illegible]

case, with modern improvements in methods of casting the concrete faces and afterward dressing them with pneumatic tools of various kinds, the concrete bridge can be made almost as handsome as the stone bridge at much less than half the cost.

The writers have heard doubts expressed by engineers as to the weathering qualities of tool-dressed concrete faces. There is ample proof that concrete surfaces, if properly proportioned and cast, may be dressed with any kind of tool, and will weather much better than sandstone, and almost as well as granite.

Fig. 1 is a view of the Connecticut Avenue concrete arch bridge, in Washington, D. C., in which the voussoirs of the arches and quoin blocks of the piers are of concrete with a mortar face about 1 in. thick, composed of granite dust and cement, the exposed faces being patent-hammered with a pneumatic tool after thorough setting. In dressing these blocks it was found that considerable time had to be allowed after they were made before they could be cut, as otherwise the dust would stick to the blades of the hammer and be driven up between them, breaking the tool.

In the winter of 1905, when this bridge was being built, the writers were residing in Washington, and paid particular attention to its construction. In the summer of 1910, in passing through the city, the photograph, Fig. 2, was taken, showing the faces of some of the voussoirs and quoins. The tool-dressed faces on these blocks were in perfect condition, and appeared as though they had been dressed only a short time.

This paper is largely a plea for an arch consisting of two ribs, rather than one with a solid soffit; with narrow rather than wide ribs, and with deep ribs of I-section rather than of rectangular section. It is also especially a plea for the three-hinged as compared with the fixed and two-hinged arches.

There are three very important advantages that the three-hinged arch has over the other types. First, in the simplicity and quickness of its analysis; second, in its adaptability to sites where rock foundation is at too great a depth to be reached; and third, the temperature stresses are entirely eliminated.

The abutments or piers of a three-hinged arch may be founded on slightly compressible material without doing any harm whatever,

whereas the slightest yielding in the abutment or pier of a fixed or two-hinged arch is sure to develop cracks in the arches.

As the three-hinged arch is merely two struts pivoted at the springing line and at the crown, and curved to follow the line of the equilibrium polygon, they will rise or fall at the crown to accommodate changes in the length of the struts due to changes of temperature.

If the bridge above the arch has been designed so that the amounts and positions of the loads on the arch ribs are known, a depth of arch rib may be assumed, its weight computed, and a force polygon and pressure line—or equilibrium polygon—drawn for the arch; the arch may then be drawn, and the stresses computed at the various joints. This entire process can be done with a slide-rule in a single day by a competent man. It very seldom takes more than three trials to fix all the dimensions of a three-hinged arch, whereas it will take several days for each analysis of the fixed arch.

The correct center line for the three-hinged arch should be drawn as follows: Assume the arch to have one-half the span fully loaded and no live load on the other half. Draw the equilibrium polygon for both the loaded and unloaded sides, then draw a line for the center line of the arch midway between the two polygons, as nearly as may be, by taking not more than three centers. It is known that the equilibrium polygons for the arch fully loaded and with no load will lie between the polygons for the half loaded and half unloaded sides; therefore, it is known that the center line drawn is for the most extreme positions of the equilibrium polygon. This may be called drawing a pressure line, and building an arch around it, and is the economical way to design an arch, as the writers can testify from experience.

The three-hinged arch has two slight disadvantages as compared with the fixed arch. First, the cost of the hinges; and second, its somewhat awkward appearance on account of the necessarily increased thickness at the haunches. The second of these cannot be avoided, but the saving in concrete in an arch of large span much more than pays for the hinges.

Fig. 3 shows two bridges which are identical in design above the arch ribs: one consists of two fixed, and the other of two three-hinged arch ribs. On Plate XV are shown cross-sections of these bridges. The stresses in the arches of these two bridges are about the same;

PLATE XIV.
TRANS. AM. SOC. CIV. ENGRS.
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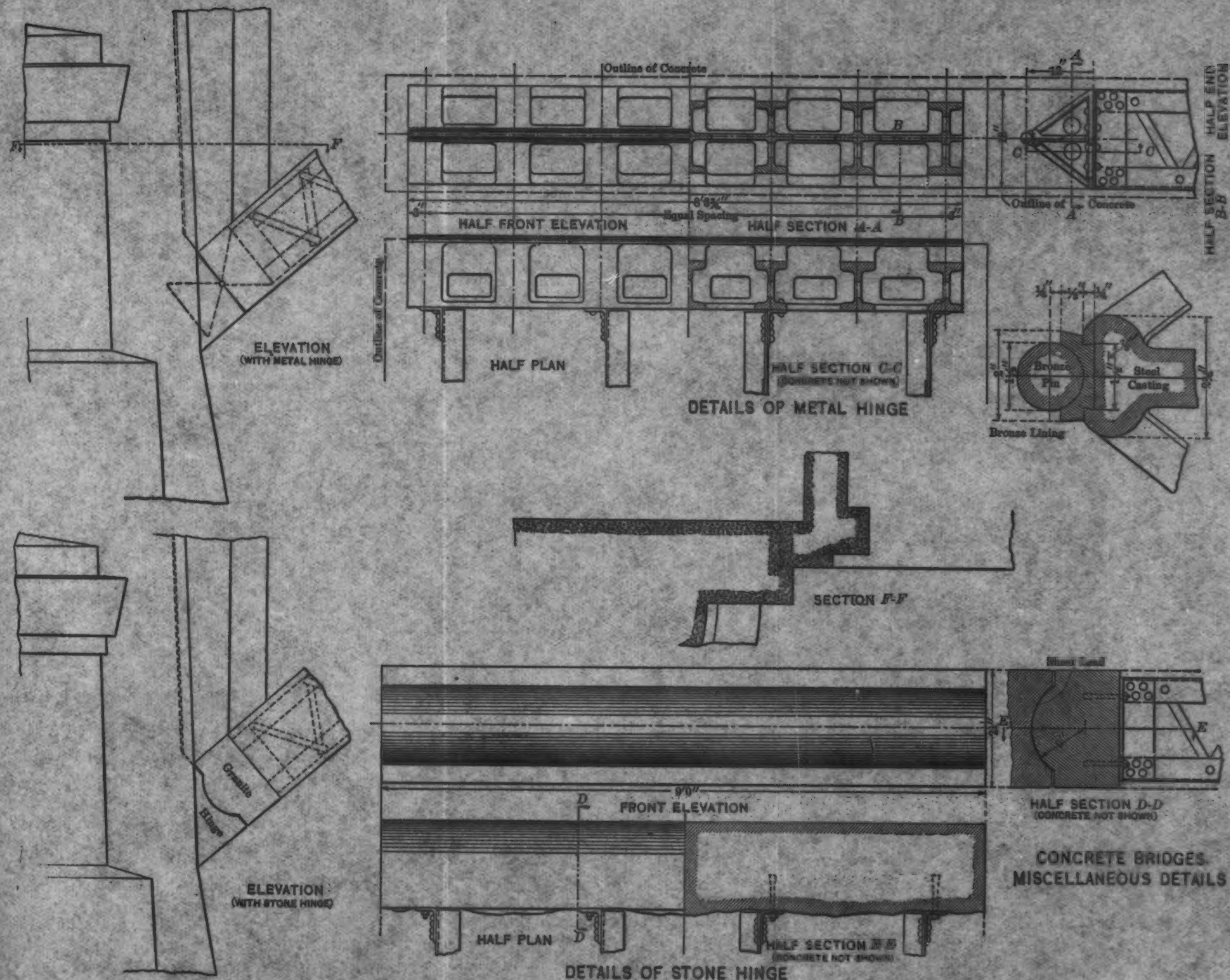
SMITH ON
CONCRETE BRIDGES.



CONCRETE BRIDGES.
MISCELLANEOUS DETAILS

that in the fixed arch, however, runs up to almost 10% more than that in the other near the springing line.

The stress from the temperature in the fixed arch here shown is in some places 40% of the total, assuming a rise and fall of 40° Fahr. from the mean temperature. An additional objection to the fixed arch is the uncertainty of temperature stresses. Conservative engineers generally assume a variation of 40° Fahr. each way from the mean. Some, however, assume a total variation of 40 degrees. The writers do not believe this to be good practice, for the reason that the time of the construction of the arches cannot be specified, therefore some may be built in quite cold weather, and others when the weather is warm. In the Rye Outlet Bridge, for example, the first arch was poured in April, when the average temperature was about 50 degrees. This arch, therefore, probably has a variation of about 40° each way from the setting temperature. The last arch was poured about July 1st, when the average temperature was more than 70 degrees. As concrete sets at a temperature several degrees above that of the surrounding air, the probable maximum drop in temperature below that of setting will be about 65 degrees. The other arches range between these two, all of them, probably, having a greater fall than rise. This uncertainty cannot be guarded against, as it is manifestly impossible to specify at just what time of the year the arches shall be constructed. If a bridge is safeguarded against this uncertainty by designing the arches for a variation of 60° each way from the setting temperature, it greatly increases the cost, and renders the three-hinged arch still more economical in comparison.

Fig. 3 shows a fixed and a three-hinged arch. Their construction, with the exception of the arch, is the same. On Plate XIV there is shown a cast-steel hinge for the three-hinged arch, similar to those used in the Traver Hollow Bridge now being built by the New York Board of Water Supply. On the same plate there is shown a granite hinge. The cost of the steel in the hinge would be about 6 cents per lb., and of the bronze pin and lining about 30 cents per lb. The cost of the granite would be about $50 per cu. yd. and the lead about 8 cents per lb. The cost of the concrete for the Rye Outlet Bridge would be about $12.50 per cu. yd. The quantity of concrete in the fixed arch span on Fig. 3 is 380 cu. yd.; that in the three-hinged arch is 200 cu. yd. The difference in cost of the two bridges, therefore, would be:

WITH METAL HINGES.

Fixed arch, excess in concrete, 180 cu. yd. at $12.50		$2 250
Three-hinged arch, steel hinges, 11 000 lb. steel at 6 cents	$660	
Three-hinged arch, bronze pin, etc., 600 lb. bronze at 30 cents	180	
		840
Difference in favor of the three-hinged arch		$1 410

WITH GRANITE HINGES.

Fixed arch, excess in concrete, 180 cu. yd. at $12.50		$2 250
Three-hinged arch, granite hinge, 9 cu. yd. at $50	$450	
Three-hinged arch, sheet lead, 600 lb. at 8 cents	48	
		498
Difference in favor of the three-hinged arch		$1 752

The ribs of this arch are rather too wide for a stone hinge, as it is not good practice to cut a thin stone 9 ft. in length, and it is not desirable to have a joint in the hinge. If the arch rib is wider than 6 or 7 ft., it is better to use a metal hinge; if it is of less width, a stone hinge may be used with advantage, as it is very simple and economical.

When the arch span is 200 ft. or more, there is a decided economy in making the rib of I-section instead of rectangular. By building the rib of this section there is a gain in two ways. A portion of the concrete is taken from along the neutral axis of the arch, where it does the minimum amount of good, and a portion of it is replaced along the upper and lower edges of the rib as flanges, where it will act to much greater advantage. These flanges should not project very far, and the slopes connecting them with the web should be quite steep, so that there will be no difficulty in filling the flanges completely with concrete. The web should be left sufficiently wide to contain two steel ribs, with room between them in which the men may move about in placing and ramming the concrete. This makes the minimum satisfactory thickness of the web about 3 ft. The flanges should not project more than 12 or 15 in. from the web; therefore, with a 3-ft. web, the width of the rib would not be more than 5 or 5½ ft.

The analysis of the rib of I-section is just as simple as that of

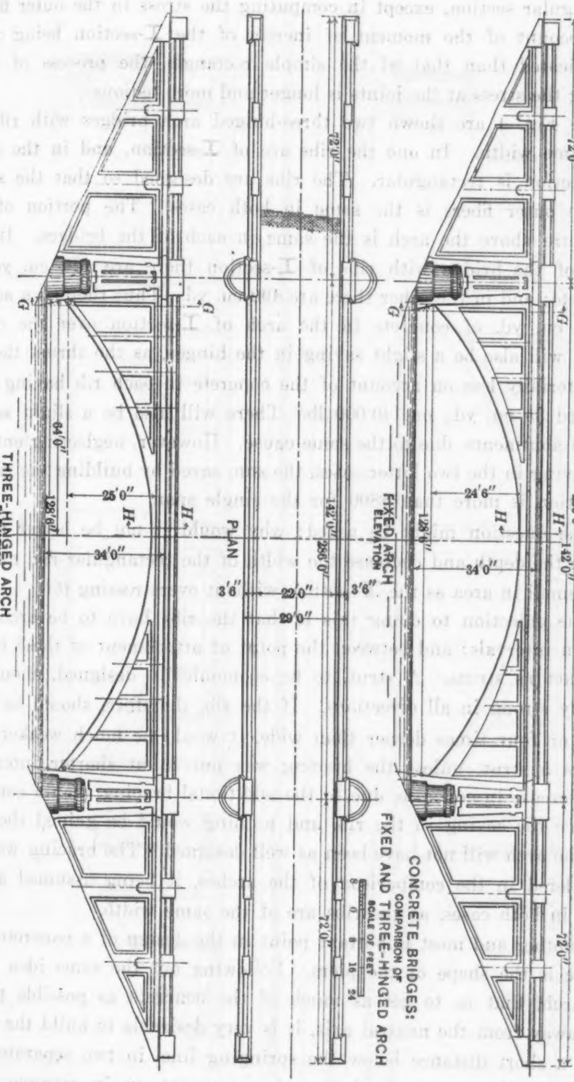

CONCRETE BRIDGES:
COMPARISON OF
FIXED AND THREE-HINGED ARCH
SCALE OF FEET
0 8 16 24
FIXED ARCH
ELEVATION
PLAN
THREE-HINGED ARCH
ELEVATION
72'0"
286'0"
142'0"
24'6"
25'0"
34'0"
22'0"
29'0"
3'6"
64'0"
G
H
H'

FIG. 8

rectangular section, except in computing the stress in the outer fibers. On account of the moment of inertia of the I-section being more complicated than that of the simple rectangle, the process of computing the stress at the joints is longer and more tedious.

On Fig. 4 are shown two three-hinged arch bridges with ribs of the same width. In one the ribs are of I-section, and in the other the section is rectangular. The ribs are designed so that the stress in the outer fibers is the same in both cases. The portion of the structure above the arch is the same in each of the bridges. In the arch of the bridge with ribs of I-section there are 420 cu. yd. of concrete; and in the other there are 490 cu. yd. Thus there is a saving of 70 cu. yd. of concrete in the arch of I-section over the other. There will also be a slight saving in the hinges, as the thrust thereon is materially less on account of the concrete in each rib having been reduced 35 cu. yd., or 140 000 lb. There will also be a slight saving in the abutments, due to the same cause. However, neglecting entirely the saving in the two latter cases, the sum saved by building the ribs of I-section is more than $800 for the single span.

The question might be asked: why would it not be better to increase the depth and decrease the width of the rectangular rib, making it as small in area as the I-section, without overstressing it?

The objection to doing this is that the ribs have to be braced at certain intervals; and between the point of attachment of these braces they act as struts. A strut, to be economically designed, should be equally strong in all directions. If the rib, therefore, should be built three or four times deeper than wide, it would be much weaker, acting as a strut, unless the bracing was put in at shorter intervals; the increase in concrete, due to the additional bracing, would counterbalance the saving in the rib, and nothing would be gained thereby; and the arch will not have been as well designed. The bracing was not considered in the comparison of the arches, it being assumed as the same in both cases, as the ribs are of the same width.

Another and most important point in the design of a concrete arch bridge is the shape of the piers. Following out the same idea as in the arch, that is, to get as much of the concrete as possible placed well away from the neutral axis, it is very desirable to build the piers, from a short distance below the springing line, in two separate legs, braced at whatever intervals may be necessary, or, in extreme cases,

connecting them by a thin diaphragm down to where they enter the ground. These legs should be made as thin as the nature of the case will allow, in directions transverse to the axis of the bridge, i.e., in the direction of the axis of the bridge, they [illegible] in order to keep the point of application of the resultant of all forces acting at every point, within the middle third, and also to keep the stress within the desired limits.

In a rectangular pier, generally, if it is desired to keep the line of pressure within the middle third, the stress will not be nearly as great as it is proper to allow, and yet it is not good practice to let the point of application of the resultant lie outside the middle third and thus cause tension in the joints. On the other hand, with the braced pier, one is able, with a little care, to design a pier which will have the line of pressure within the middle third, and also have the stresses about the figure desired, thereby attaining a pier of maximum economy.

On Plate XV there are two piers illustrating this point. The first is a braced pier somewhat similar to those in the Big Outlet Bridge. From a short distance below the springing line the pier consists of two separate legs, braced at two points. This pier is of about the same height as two of the piers in the Big Outlet Bridge. The second is an ordinary solid pier of rectangular section of the same height. Both piers are designed so that the pressure line is at the middle third point of all joints. On comparing these two piers it will be seen that the stresses in the braced pier remain quite uniform, except near the bottom, where they increase a moderate amount. In the upper part of the rectangular pier, however, the stresses are too low for economy, and yet it is not desirable to increase the stress by allowing the pressure line to go outside the middle third and cause tension in the joints. The difference in the quantity of material in these two piers is surprising; the braced pier has only 1 785 cu. yd. of concrete; the solid pier has 2 825 cu. yd., a difference of 1 040 cu. yd., or 58% of the braced pier and 37% of the solid pier. The difference in cost of the two piers, therefore, is about [illegible]. The braces in the pier, however, are of reinforced concrete, which tends to decrease the difference somewhat; but, on the other hand, if the earth excavation is deep, and within a cofferdam, there will be much less excavation for the braced than for the solid pier, which increases the difference in the cost. Piers of this shape are not handsome, but if economy is

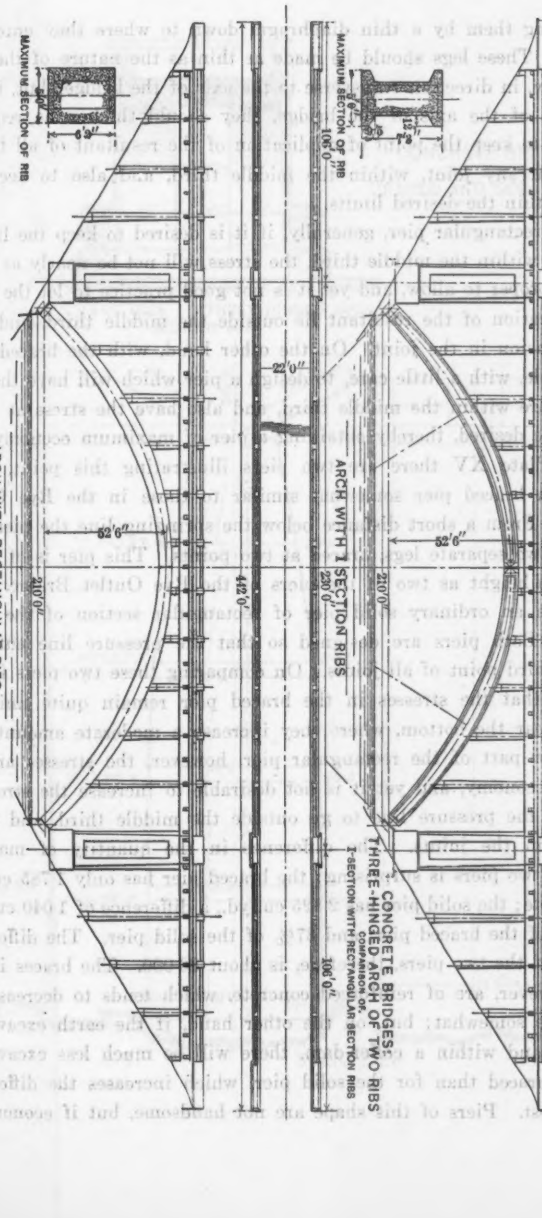


FIG. 4.

connecting them by a thin diaphragm down to where they enter the ground. These legs should be made as thin as the nature of the case will allow, in direction transverse to the axis of the bridge; but, in the direction of the axis of the bridge, they should thicken as required in order to keep the point of application of the resultant of all forces, acting on any joint, within the middle third, and also to keep the stress within the desired limits.

In a rectangular pier, generally, if it is desired to keep the line of pressure within the middle third, the stress will not be nearly as great as it is proper to allow, and yet it is not good practice to let the point of application of the resultant lie outside the middle third, and thus cause tension in the joint. On the other hand, with the braced pier, one is able, with a little care, to design a pier which will have the line of pressure within the middle third, and also have the stress at about the figure desired, thereby obtaining a pier of maximum economy.

On Plate XV there are two piers illustrating this point. The first is a braced pier somewhat similar to those in the Rye Outlet Bridge. From a short distance below the springing line the pier consists of two separate legs, braced at two points. This pier is of about the same height as two of the piers of the Rye Outlet Bridge. The second is an ordinary solid pier of rectangular section of the same height. Both piers are designed so that the pressure line cuts the middle third point of all joints. On comparing these two piers it will be seen that the stresses in the braced pier remain quite uniform, except near the bottom, where they increase a moderate amount. In the upper part of the rectangular pier, however, the stresses are too low for economy, and yet it is not desirable to increase the stress by allowing the pressure line to go outside the middle third, and cause tension in the joints. The difference in the quantity of material in these two piers is surprising; the braced pier has only 1 785 cu. yd. of concrete; the solid pier has 2 825 cu. yd., a difference of 1 040 cu. yd., or 58% of the braced pier, and 37% of the solid pier. The difference in cost of the two piers, therefore, is about $6 000. The braces in the pier, however, are of reinforced concrete, which tends to decrease the difference somewhat; but, on the other hand, if the earth excavation is deep, and within a coffer-dam, there will be much less excavation for the braced than for the solid pier, which increases the difference in the cost. Piers of this shape are not handsome, but if economy is

PLATE XV.
TRANS. AM. SOC. CIV. ENGRS.
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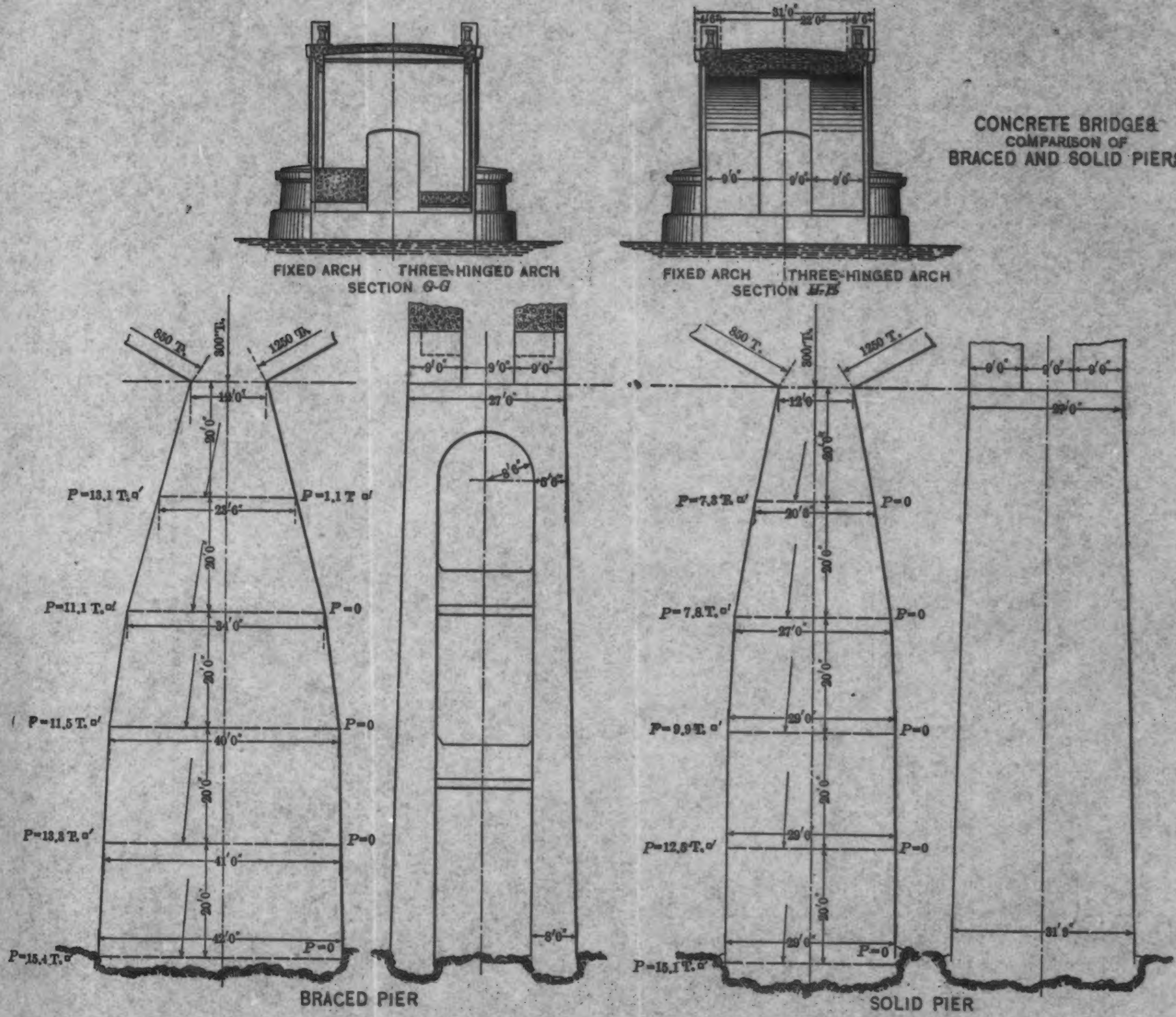
SMITH ON
CONCRETE BRIDGES.

CONCRETE BRIDGES
COMPARISON OF
BRACED AND SOLID PIERS



an important consideration, they are very desirable; and if they are to be hidden by the water from a short distance below the springing line, as in the Rye Outlet Bridge, they will not detract from the appearance.

This type of pier is especially economical for a bridge in running water in which there is likely to be heavy blocks of ice or logs of wood moving rapidly. By putting a cut-water on the up-stream side to deflect such objects, and spreading the legs of the pier rapidly and connecting them with a thin diaphragm, the bridge can be braced thoroughly with very little additional masonry in the piers. In a very cold climate, where heavy ice is likely to form about the piers, the connecting diaphragm should contain steel reinforcement.

In all the bridges on the accompanying illustrations the width is the same; therefore a comparison of the quantity of concrete per linear foot of arch gives a fair idea of the economy of the arches alone. The quantity of arch concrete per linear foot of span in each of the bridges shown is as follows:

Fig. 3, fixed arch, ribs 9 ft. wide	3.00 cu. yd.
Fig. 3, three-hinged arch, ribs 9 ft. wide	1.57 " "
Fig. 4, three-hinged arch, rectangular ribs, 5 ft. wide	2.33 " "
Fig. 4, three-hinged arch, I-section ribs, 5 ft. wide	2.00 " "

From this it is seen that, with the narrow ribs, although the abutments are 50% farther apart, there is very little more concrete per linear foot than in the shorter three-hinged arch with ribs 9 ft. wide.

The writers think that the following conclusions are amply justified by the foregoing comparative investigations:

First, that an arch span consisting of two separate ribs is more economical than one with a solid soffit, if the span is greater than 100 ft.

Second, that narrow, deep ribs are more economical than thin, wide ones.

Third, that the three-hinged arch is more economical and reliable than fixed or two-hinged arches for spans greater than 100 ft.

Fourth, that for spans of 200 ft. or more, the rib of I-section is more economical than the rectangular rib.

Fifth, that piers of any considerable height are much more econom-

ical if built of two separate legs thoroughly braced, thickening rapidly in the direction of the axis of the bridge as they go down.

All numerical computations have been omitted herein, as all the processes of investigation are simple, and it was not desired to lengthen the paper by including them.

The ratio of the modulus of elasticity of steel to concrete was taken as 15, and in obtaining the moment of inertia of the sections at the various joints the steel was considered as being replaced by 15 times its area of concrete.

The Rye Outlet and Traver Hollow Bridges were constructed by the Board of Water Supply, consisting of Messrs. Charles Straus, Charles N. Chadwick, and John F. Galvin. J. Waldo Smith, M. Am. Soc. C. E., is Chief Engineer and A. D. Flinn, M. Am. Soc. C. E., Department Engineer, Headquarters Department, in charge of all design. Charles E. Gregory, M. Am. Soc. C. E., was Designing Engineer in Charge of Design of Dams and Bridges, and the senior writer had charge of the design of the Rye Outlet Bridge and the preliminary design of the Traver Hollow Bridge under Mr. Gregory.

DISCUSSION

S. W. BOWEN, M. AM. SOC. C. E. (by letter).—The writer is much interested in this paper, especially in that portion relating to three-hinged ribbed arches. The comparisons of cost of the three-hinged and fixed types seem to indicate that the former is not such an expensive form of construction as we are sometimes led to believe. The comparison, however, is incomplete, as the cost of the rib reinforcement is not included. A fair comparison of the two types cannot be made without including this item. Mr. Bowen.

As pointed out by the authors, the three-hinged arch possesses a number of advantages over the fixed type, not the least of which is the certainty and simplicity of the calculations. One cannot carry through even the more simplified theories of fixed-arch design without being impressed by the length and complicated nature of the computations involved. These computations are based on assumptions which may not be strictly correct and may be entirely upset by a slight movement of one of the supports, to say nothing of the complications caused by temperature. In comparison with this, there is little, if any, uncertainty in the design of a three-hinged arch. The computations are simple, and the stresses in the ribs can be ascertained as accurately as the dead and live loads.

In some reinforced concrete viaducts, designed recently by the writer, cast-steel hinges having hemispherical, ball-and-socket joints were used. The price paid for these castings was 7 cents per lb. in place, which is high. Even at this price, the cost of the hinges amounted only to about $3\frac{1}{2}\%$ of the cost of the structure. This does not seem to be an extravagant price, considering the many advantages obtained. The writer believes that more can be gained by developing a cheap form of hinge than by producing new theories of fixed-arch design.

With reference to the ribbed type, as compared with the solid arch ring or barrel type, the former has one advantage over the latter which is not brought out by the authors: This is the fact that no water-proofing is required where ribbed arches are used. Water-proofing is frequently an expensive and troublesome proposition, and the type of construction that does away with it is worthy of consideration for that reason alone.

As to the I-shaped section for the ribs of long spans, it is doubtful whether this cross-section is much more economical than the rectangular section. The forms for the former will be more expensive per cubic yard of concrete than those for the latter, and the cost of placing concrete in the I-section will be greater per cubic yard than in the rectangular section. This will tend to reduce the difference in cost due to the smaller yardage of concrete in the I-section.

Mr. Bowen.

In conclusion, the writer wishes to state that he agrees fully with the authors as to the advantages of the three-hinged ribbed arch over other types of arch, and believes that this valuable paper will do much toward bringing this type into more general use.

Mr. W. F. Smith.

WILSON FITCH SMITH, M. AM. SOC. C. E—Referring to the authors' arguments in favor of the three-hinged arch and the subsequent discussion of its merits, it may be of interest to recall the masterful way in which the late George S. Morison, Past-President, Am. Soc. C. E., used the principles of arch design by applying the theory of the three hinges to a masonry arch of long span.

In 1900 Mr. Morison designed a masonry highway bridge of five arches, each with a span of about 180 ft. The arches were circular segments having a rise of one-quarter of the span. The bridge was 80 ft. wide and the arches extended for its full width. The arch ribs varied in depth from 5 ft. at the crown to 7 ft. at the springing line, and were to be of limestone with voussoirs of full depth on the faces of the arches. The arch rings carried cross-walls 4 ft. thick, spaced about 15 ft. apart, which, in turn, carried full centered arches supporting the floor, except at the three center panels where the spandrel was carried up solid to the floor level. At the crown joint and springing line joints were inserted lead bearings, about 1 ft. wide at the crown and 18 in. wide at the end joints. These plates were to act as hinges, permitting an adjustment of the arch under the dead-load strains, and the joints were to be filled with cement mortar after the completion of the bridge.

In the design of these arches the usual method was followed of dividing the arch ring into short sections, considering the weight of each as acting in a vertical plane through the center of gravity of each section. The weight of the spandrel arches and flooring was divided in a similar manner. The strains were determined graphically, and by leaving voids of various sizes in the concrete filling over some of the spandrel arches and loading others with pig iron embedded in the concrete, an arrangement of loads was obtained which produced a resultant curve of pressure passing through the center line of the arch ring at each panel point under the cross-walls and at the hinge joints.

For various conditions of live load, the arches were considered as fixed (the hinge joints being filled before admitting traffic) and the resultant lines of pressure lay well within the middle-third of the arch rings, giving very moderate pressures for such large spans.

This treatment, in its simplicity and the skillful use of the three-hinge theory, is an example of the clear-sighted manner in which Mr. Morison approached problems of design, and shows the attributes of the great engineer.

C. E. Gregory, Assoc. M. Am. Soc. C. E.—The speaker was the Designing Engineer in charge of the design of bridges built by the Board of Water Supply, as referred to in this paper, and wishes to make a few statements to amplify the relation of the work as done by the Board of Water Supply to this paper, and to put others who were concerned in the design of these bridges in a right position in regard to it. The first point he desires to make is this: Mr. Gregory.

The authors reach the conclusion that in practically all cases the three-hinged arch is more economical for long spans than the ordinary fixed-end arch. When designing the Traver Hollow Bridge, the speaker did not, and does not now, concur in such a conclusion. The three-hinged type was selected largely because of expected slight settlement in the foundations, rather than the economy of the type of bridge. The preliminary estimates indicated that a three-hinged arch would be more expensive than a fixed-end arch.

The economy of the divided pier is brought out by the authors, but the manner in which the design of the piers of the Rye Outlet Bridge was arrived at differed considerably from that described in the paper, although meeting the assumptions of the authors. The interesting and unusual feature of this bridge is that it is an arcade of five reinforced concrete arches supported on exceptionally high piers. It was found that the yielding of the top of a high pier of ordinary shape under the unbalanced live load arch thrust (the height of the highest pier in this bridge being a little more than 100 ft.) would be sufficient to create considerable stress in the arch without exceeding the usual limits of stress or creating tension in the pier. The design was made to limit the yielding of the piers to that which could be safely and economically provided for in the arches. The entire arcade was treated as a complete elastic structure, and determination was made of what proportion of the unbalanced thrust from a loaded arch would be carried by the adjacent pier and by other arches and piers of the arcade. The shape and size of pier adopted was found to be necessary from calculations made as just outlined.

This work was done under the speaker's supervision by Mr. George L. Bennett and Orrin L. Brodie, M. Am. Soc. C. E., who were at the time assigned to Mr. Smith's staff, but worked on this problem under the speaker's immediate direction.

Henry H. Quimby, M. Am. Soc. C. E.—Experience in the design of concrete arches confirms the opinion expressed by Mr. Gregory regarding the relative economy of fixed and hinged rings, and the justification for the use of hinges. Computations for comparison of designs in several cases have failed to show any saving in cost of a three-hinged type over a fixed type when the ratio of rise to span was fairly good. Hinges are justified only when the foundations are unsatisfactory, or in cases where the arch is very flat. In Mr. Quimby.

Mr. Quimby.

a flat arch the crown movement due to change in length of ring from temperature is greater than in a high-pitched one, and the temperature stress may thus become great enough to warrant the use of a hinge to avoid it. The arches shown in the paper, however, are not flat, but have a good rise.

The authors give estimated quantities of concrete required for the arch rings in the two designs shown, and represent the fixed type as requiring 90% more concrete than the hinged. This is hard to understand. A fixed ring should require only a little more than a hinged ring. The depth at the crown must be about the same for each because the horizontal thrust is about the same, and any theoretical eccentricity, due to unsymmetrical loading, in the fixed ring will probably be fully matched by the effect of friction of the hinge in the other case, for the inevitable deflection caused by the weight of the floor imposed after the closure of the arch will be likely to produce the effect of eccentric thrust. At the quarter points the hinged ring will need considerably more depth than the fixed ring, and it will be only at the springing line that the fixed ring will be deeper and require more concrete. As the cost of effective hinges is quite considerable, the net result is generally the reverse of that reported in the paper.

If it be said that the greater depth of the fixed ring design in the paper was made because of temperature stress, the reply is that temperature stress necessarily increases with the depth of the ring, and adding to the depth because of such stress is somewhat like piling a load on a horse's back to enable him to pull a heavier wagon—the poor legs may be overtaxed.

It would seem, therefore, that there must be something wrong, either with the estimates or with one of the designs.

The stone hinges shown in the paper give the impression of being likely to develop friction if they work at all, and this will entail eccentricity of stress in the arch ring, to that extent detracting from the only advantage of the hinge.

The matter of appearance of an arch is important in many places, and the bulging at the quarter that is characteristic of the ordinary hinged ring is distinctly ungraceful and displeasing to the miscellaneous eye which cannot understand the reason for it, while the eye of the designer who knows that the point of greatest shear is the point of greatest single stress, desires to see the arch thickest at the spring where that stress is the greatest.

One very important consideration in a discussion of stress in a masonry arch is the method of construction, and it is generally far more vital than temperature fluctuations. The old-fashioned method of laying up an arch ring from the springing line to the crown, with the attendant progressive deformation of the centering and consequent cracking of the earlier portion of the ring practically reduces the work-

ing thickness of the ring to a fraction of that designed, and may be the real cause of some failures that have occurred. The proper way to avoid cracks or initial stresses in the arch ring due to settlement or compression of the centering during construction is to build the ring in voussoir blocks with narrow key spaces between them. Then when all the blocks are made, and the centering has, therefore, nearly its full load, after the blocks have seasoned and shrunk, cast the keys. Of course, if the arch is small enough to permit the casting of the full length of the ring in one day—say within 10 hours—the separation into blocks will not be needed, because the first portion cast will be still plastic enough when the ring is closed to adjust itself to the deformed centering. It is impracticable to eliminate initial stress wholly, for any of the dead load that is added subsequently to keying will cause some deformation of the ring and consequently initial stress somewhere; but this should never be enough to be serious. **Mr. Quimby.**

The claim advanced in the paper that concrete will weather better than good sandstone, and almost if not quite as well as granite, is hardly to be expected from an experienced observer of concrete work. If the authors know of any formula that will produce concrete with such a desirable quality, they should give it to the Profession. It is a poor quality of building stone that will not hold its surface and its edges better than the best concrete. The statement in the paper sounds a little over-enthusiastic. The friends of concrete will promote its use more effectively by avoiding extravagant claims for it.

Another point in the paper that seems not entirely clear is the claim of marked advantage in the I-shape of arch rib. The arch ring is a column—not a beam. If the curve be true and the line of thrust be central, there is absolutely no beam action. Of course, the width—horizontal thickness—of the rib, and its depth or vertical thickness, should both be theoretically adjusted to make a column that will fail by crushing rather than by buckling, and this will require some minimum width. As regards lateral buckling, any given width is as effective at the mid-depth as at the flanges, and, in any case, suitable transverse bracing should be provided. The method of estimating the saving due to the coring out of the concrete at the sides is hardly fair, because a very large part of the cost of such concrete is in the support of it, and any increase or decrease in quantity will be proper to be charged at only the cost of the material and placing. Besides, the extra cost of the additional form work necessary for the paneling will materially reduce the seeming economy in material.

Replying to the question of a member as to the observed actual range of temperature in concrete arches: Two bridges, constructed within the past 5 years, with electrical resistance thermometers embedded at the mid-line, show temperatures in the concrete ranging far less than is usually assumed. The depth of embedment in one case

Mr. Quimby. is 2 ft. 6 in. and in the other, 4 ft. 9 in. The former is a spandrel filled arch and the other is an open spandrel. The maximum range of temperature recorded thus far has been from 31° Fahr. in winter to 73° in summer, and measurements of change in elevation of crown made at the same times show a rise and fall computed to correspond to a change of temperature throughout the arch ring of 45° Fahr. between the extremes of cold and heat.

Mr. Janni. A. C. JANNI, M. AM. SOC. C. E.—The statement, among others, made concerning the quickness, simplicity, and certainty of the design for three-hinged arches, deserves careful investigation, and should not be accepted off-hand, or as a matter of course.

Therefore, it is not without interest to recall what both theory and experience teach with reference to the equilibrium of this particular kind of construction, and to ascertain to what extent a designer may trust his results.

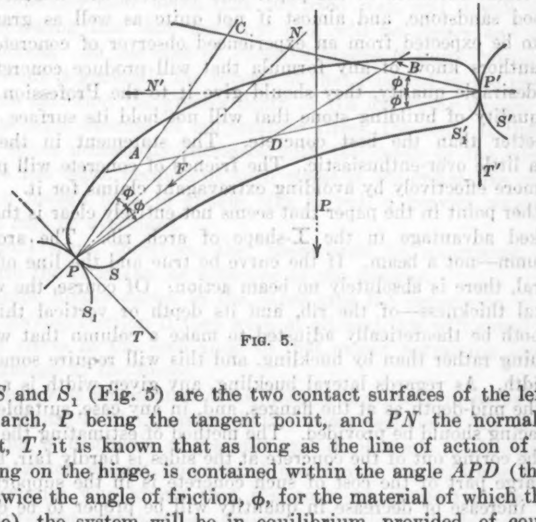


FIG. 5.

If S and S_1 (Fig. 5) are the two contact surfaces of the left hinge of an arch, P being the tangent point, and PN the normal to the tangent, T, it is known that as long as the line of action of a force, F, acting on the hinge, is contained within the angle APD (this angle being twice the angle of friction, ϕ, for the material of which the hinge is made), the system will be in equilibrium, provided, of course, the force, F, is a compressive one with respect to the tangent, PT. Similar remarks may be made regarding the surfaces, S' and S_1', of the hinge at the key.

Therefore, as long as the two components of a force, P, acting on the system, will keep within the two angles, APD and $BP'D$, the system itself will be in equilibrium, and the points, A and B, are to be regarded as the extreme limits of this equilibrium position.

In other words, the system will be always in equilibrium under the action of any force the line of action of which cuts the area, $ACBD$. Mr. Janni.

The foregoing demonstration is very plain, but its foundation is rather indefinite; in fact, it is all based on the assumption that the angle, ϕ, is a known quantity, and, even if that is true, there are two solutions, according to the theory, that satisfy the conditions of equilibrium and, especially for an arch of large and flat span, give quite different results.

Concerning the friction laws, it can be said that, outside of those values of ϕ reported in all engineers' handbooks, which for every-day purposes are tolerably reliable, those laws are not well known at present.

The history of the researches on friction is very brief. Amontos, in 1699, started for the first time a research on the friction of some machine of his, without, however, throwing any important light on the general matter.

In 1781, Coulomb took up the researches on this important problem and announced to the world his results, giving out laws on friction, as well as values of ϕ for different materials.

In 1785, Vince started his researches on the same question and rejected, as unreliable, Coulomb's laws and values of ϕ.

In 1829, G. Rennie, who made tests under about the same conditions as Coulomb, found more or less similar results.

Finally, in 1831, Morin presented before the "Académie des Sciences", in Paris, the results on his friction tests, and, though he confirmed Coulomb's fundamental laws, he found numerical values for ϕ which were very different. It is worthy of note that, though Morin made his tests on a larger scale than Coulomb, he did not go farther than a pressure of about 5.2 lb. per sq. in.

As far as the speaker knows, these are the only works on the subject which stand high on account of the illustrious names of their authors; unfortunately, they do not agree among themselves.

Modern investigations, however, without adding anything definite, have found that the friction law, as given by Morin,

$$R = fN$$

where: R = Resistance due to friction;

N = Normal component of the force acting on the body; and

f = A numerical coefficient depending on the materials in contact;

is not complete; because, though f is dependent on the materials in contact, it is dependent also on the speed with which one surface moves on the other and on the intensity of the acting force; these laws, however, have not yet been determined.

Mr. Janni. The only thing known at present is that the value of ϕ, at the start, is between 1.5 and 3 times its value during the movement, and this is quite a wide range when one considers the certainty of results.

Usually, the hinges are metallic, or, even in stone, they have a shape which might be regarded as a hinge; the problem then is solved, as is well known, in the following manner:

Let A (Fig. 6) be the left hinge of an arch; drawing the friction circle with the radius r sin. ϕ (where r is the radius of the hinge and ϕ the angle of friction), it is known that all forces having lines of action which cut that circle will not move the hinge about its center, and all forces having lines of action tangent to that circle will be on the limit position of equilibrium, provided each of these forces has a tendency to compress the hinge against its groove.

If B is the hinge at the crown, it is known that the four common tangents to the two friction circles are limit positions of equilibrium, therefore, according to graphic statics, the problem already admits of four solutions.

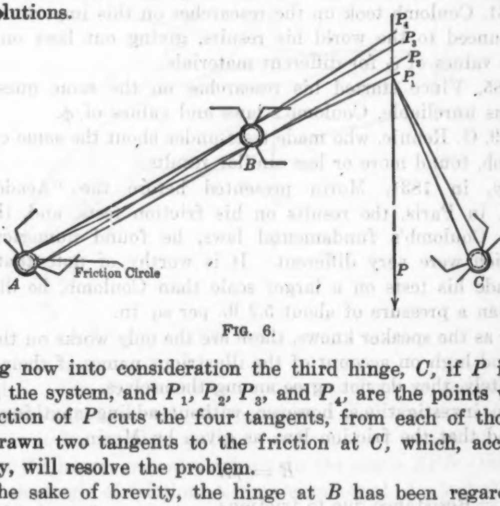


FIG. 6.

Taking now into consideration the third hinge, C, if P is a force acting on the system, and P_1, P_2, P_3, and P_4, are the points where the line of action of P cuts the four tangents, from each of those points can be drawn two tangents to the friction at C, which, according to the theory, will resolve the problem.

For the sake of brevity, the hinge at B has been regarded as of the same kind as those at A and C.

Apart from the several solutions given by the theory on the equilibrium of this kind of construction, here, too, the indeterminations, resulting from friction, must be considered.

The friction circles can be drawn quite accurately for machinery, or when it is a question of pieces well prepared for laboratory tests, for which test after test has been made so that the whole question is somewhat settled.

Concerning the friction in bridge hinges actually in operation, however, on which the pressure is often so great as to wear off the material at the contract surfaces, it is to be said with regret that there has not been a single test known to the speaker. Mr. Janni.

It can be seen, therefore, that the quantity, r sin. ϕ, has not, by any means, a known and determined value; in fact, it is rather arbitrary. In other words, the set of four points, P_1, P_2, P_3, and P_4, can be substituted for another set of four points, provided they satisfy the requirements of graphic statics.

Although all the stated indeterminations (and there are others) do not prevent a designer from patching up some kind of design for a three-hinged arch, it is not possible to accept the statement concerning the reliability of such design.

The hinges, however, in a concrete or masonry arch can be used with advantage sometimes during the construction of the arches, when there is no steel reinforcement to take up the tension at the springing line and at the key, due to a possible excessive settlement of the centering; then, after the centering has been removed, the hinges should be concreted. Thus the arch behaves as a three-hinged arch, as far as the dead load is concerned, and as a rigid arch for the live load and temperature. In that instance the hinges, free from the enormous pressure of the dead load, can easily accompany any movement of it, being concreted afterward, when their behavior begins to be extremely doubtful.

The granite railroad bridge at Morbegno, Italy, with 229.65 ft. span and 32.8 ft. rise, has been built in accordance with the foregoing method.

In comparing concrete and stone bridges, the authors rightly state that one of the advantages of the former is that it is stronger and much more reliable, because of the absence of joints; therefore it is not clear why the paper is a special plea for the three-hinged arch, which has three joints, as compared with the rigid arch, which has none.

Concerning the statement that the trial design for a fixed-end arch takes several days for each analysis, it can be said that it takes no longer than the trial design for a three-hinged arch, and possibly less.

The stresses in the arch with fixed ends, due to dead load and temperature, can be calculated in a day's work easily.

A new method of designing an arch, based on the rather recent theory of the "Ellipse of Elasticity", is described in a paper by the speaker.* This is a purely graphical method and holds, irrespective of the geometrical form of the axis of the arch, irrespective also of the law of variation of the cross-section of the latter, and of the assumptions of loading. In other words, it is possible, by that method, to draw the lines of influence for moments, for any section

* *Journal*, Western Society of Engineers, May, 1913.

Mr. Janni. of the arch, before the loads are applied on the arch, and, as a consequence, the most prejudicial hypothesis of loading for the chosen section is given at once, without any trial.

In that paper, in addition to the theoretical part, the speaker showed a practical application to an arch which he had designed and built by that method.

In contrast with the unreliability of the results in a design for a three-hinged arch, it is not without interest to relate that for a fixed-end reinforced concrete arch, built in France, having a span of 318 ft., the difference between the stresses calculated and the actual stresses reached a maximum of 14.7 per cent.

Every engineer fully appreciates the importance of these results, especially when he considers that, in steel bridges of the best design, that difference reaches 25%, and, when designed on the assumption that there is no friction at the joints, the difference may be more than 100 per cent.

The horizontal thrust, due to temperature changes, is directly proportional to the coefficient of expansion, to the change of temperature in degrees, to the modulus of elasticity of the material, and to the span (measured at the spring points of the geometrical axis of the arch); but it is inversely proportional to the inertia moment of the whole elastic arch with respect to the horizontal (if the arch has its spring points at the same level) passing through a certain point called the center of gravity of the elastic arch.

If, for the same span, arches with different rises are drawn, it can be seen that, although the factors directly proportional to the horizontal thrust remain unchanged (the variation of the span of the geometrical axis may be disregarded), the moment of inertia of the arch increases rapidly with the increase of the rise of the arch; so that the effects of a change in temperature diminish in importance with the increase of the rise.

The shrinkage of the concrete due to setting and that due to the dead load are two important causes of stress to be taken care of, especially the former. If the arch is very flat and has a large span, the best and most economical way to take care of the shrinkage due to setting is by neutralizing its effects, as much as possible. A span of 170 ft. with 17 ft. rise, and two spans of 140 ft. with 14 ft. rise, designed by the speaker, have been built, breaking the arches into voussoirs each about 20 ft. long, and leaving between each two consecutive voussoirs a small voussoir, or key, of 2 ft. The voussoirs were cast first and the keys were not cast until the last concrete of the voussoirs was 10 days old, thus avoiding, for the 170-ft. span, for instance, important tensions on the intrados at the key and on the extrados at the springing line, especially during the winter, due to a shortening

of $\frac{1}{8}$-in. of the geometrical axis of the arch, which otherwise would have taken place. Mr. Janni.

If, as the authors state, an average pressure of 12 000 lb. per sq. in. on the crown hinge of that arch was calculated, it must be inferred that the maximum pressure on the hinge was about 24 000 lb. per sq. in. at the theoretical point of contact with its groove, and it may be assumed that it was reduced to zero at each end of the groove.

The question then arises: At such a vital point of a bridge may the material be submitted to work at a rate which is never allowed for such material at other much less important points; and that without discussing the practical inadvisability of trusting on a $2\frac{1}{2}$-in. space the stability of such a comparatively enormous mass of material?

In dealing with such pressures, however, there is no longer any question of the possibility of assuming that at that point there is a hinge, ready to work; it is no more a matter of design, but of guesswork.

When a hinge of such small diameter is placed at the key, a malicious attempt to wreck the span could be successfully carried out.

Another kind of hinge—fortunately, very seldom used—is a ball-and-socket joint of cast steel consisting of a hemispherical protuberance fitting into a hemispherical cavity. Any horizontal displacement of the crown of the arch, for a structure of this kind, cannot be contemplated seriously; there is no reason for using such a hinge. In the construction of a three-hinged arch, the use of a ball-and-socket hinge means simply the addition of more friction at that joint.

Having an arch with only two ribs, and that arch hinged with the ball-and-socket type, then, at each joint cross-section there will be at least two hinges. Admitting, for the moment, that a horizontal displacement of the crown might take place, it will be admitted also, that it will be very small.

On account of the rigidity of connection between the two ribs, however, there is no possibility of a horizontal rotation of both hinges, even disregarding friction and admitting, of course, the elastic deformation of the whole system; but the effect of that displacement will be only to decrease the pressure on one hinge at the expense of the other.

However, as the use of a hinge of that kind shows that the designer assumed the possibility of a really tangible horizontal displacement of the key and the consequent horizontal rotation of the hinge, it is not without interest to ascertain what would happen to the arch if a horizontal rotation took place at the key, no matter how small it might be, provided, however, that it be a tangible quantity.

Assume that the hinge has a horizontal rotation of only 10′, and that the two hinges are 30 ft. apart; then, while one hinge presses in its socket during a rotation of 10′ (always disregarding friction), the other would be lifted out of its socket a distance of about 1 in. Although

Mr. Janni.

this will not happen, on account of the elastic deformation of the system, it is fair to admit that, at that joint, the horizontal thrust is zero (without speaking of the pressure concentrated on the rotating hinge).

What happens at the key, according to the designer's assumption, will happen also at the springing line, but the order will be reversed. Then the question arises: Did the designer calculate the stresses in his elastic system under such abnormal conditions of equilibrium? If he did, it is certain that he will never advocate the ease, speed, and certainty of a design for a three-hinged arch, especially with ball-and-socket joints; if he did not, then his enthusiasm for structures of this design is quite natural.

M. Mesnager, wisely concerned about the secondary stresses arising from the use of the hinges in arch bridges, and for which there is no way, at present, to determine their exact amount, thought that a half-rigid joint "semi-articulation", the working behavior of which could be easily determined, would be preferable to the usual hinges.

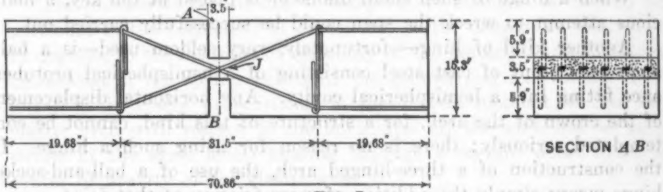


FIG. 7.

For that purpose, several years ago, under his personal supervision, the Testing Laboratory of the "École des Ponts et Chaussées" started researches on a joint, Fig. 7, built in reinforced concrete according to his plans. Without going into details concerning his tests, it is enough to repeat one of his remarks:

"Le moment qui a été nécessaire pour fair fléchir la semi-articulation quand elle était noyée dans le béton, n'était pas négligeable."

Then he concluded by saying that it was possible, perhaps, to free the joint, J, from concrete and fill the remaining space with some kind of non-acid asphalt.

Evidently, a hinge of this kind, if developed properly, could not be expensive, and, at the same time, would eliminate the secondary stresses depending on friction, at least.

Concerning the saving of material by building the ribs of I-shaped section, it may be remarked that such a saving, in a general way, does not mean, necessarily, a saving of money. It would be rather absurd to think that a marble statue costs less than a block of the same material having the same external dimensions, only because the actual volume in the statue is less than that in the block. In addition, how-

ever, to all doubts expressed by other speakers concerning the cheapness of an I-shaped rib, as compared with the rectangular rib, another remark may be made: The three-hinged arch, on account of its lack of rigidity, requires a stronger system of stiffeners than that needed by a fixed-end arch, and the quantity of concrete in the stiffeners for the former must, necessarily, be greater than in those for the latter; therefore the statement advanced by the authors, that the quantity of concrete was the same in each case, cannot be accepted. Mr. Janni.

In conclusion, it is to be noted that the statement concerning the complication of design for a fixed-end arch and the ease with which such an arch can be upset by a slight movement of one of its supports, is not altogether justified. This complication of design, so much complained of, is justified only if the designer expects to calculate the arch as simply as he can pick out, from the "Carnegie Handbook," the I-beams for a floor.

Concerning the latter remark (which was correct several years ago) it can be said that, by the theory of "virtual work" or, more elegantly and speedily, by the method of the "ellipse of elasticity", the designer can easily take care of such dreaded displacement of support, and even of a yielding of the soil on which the footings rest.

There is no doubt that the design for an arch is one of the most delicate problems of engineering; but, after all, it may be done without any great trouble if the designer is sufficiently familiar with his work.

The theories used are not monopolized by a few persons, but are quite accessible to any one of training and application. Fortunately for the designer, those theories cannot be reduced to "rule-of-thumb", for several reasons. If an engineer, satisfied with what he was taught at school, does not keep in touch with the daily progress of science, he runs the risk, nowadays, of being behind the times. If, for instance, in designing an arch, he still assumes first that the arch is entirely loaded and then, that it is half loaded and half unloaded—still believing that, in this way, he ascertains the maximum stresses in that arch—he is flatly wrong; those assumptions were made several years ago, because there was then no means of ascertaining, conveniently, the most prejudicial hypothesis of loading; at present, however, that method of computing stresses is antiquated as well as inadequate.

PHILIP AYLETT, ASSOC. M. AM. SOC. C. E. (by letter).—In America, the ribbed arch, especially the three-hinged type, has not received that recognition of its merits which it deserves. Its economy and advantages are better known and more highly appreciated in Europe, where it has been used extensively under almost all conditions of site and traffic, and in great variety of form and design. Mr. Aylett.

Comparatively few concrete bridges of the rib type proper (hinged or fixed) have been built, as yet, in America, and these only within

Mr. Aylett. the past decade. The literature on the subject, therefore, is comparatively scant.

The authors of this paper deserve the thanks of all interested in the use and development of this type in America, where its future is so promising and its field so extensive.

The use of three-hinged arches on compressible material is fully justified by ample experience and precedents. The foundations of abutments and piers in such cases, however, demand more careful investigation, study, and design. A fixed or a two-hinged arch may also often be built with entire safety on compressible material, provided the foundations are properly designed and built, including the use of batter piles and other methods of taking up thrust. This also is justified by numerous structures under such conditions now extant, both in Europe and America.

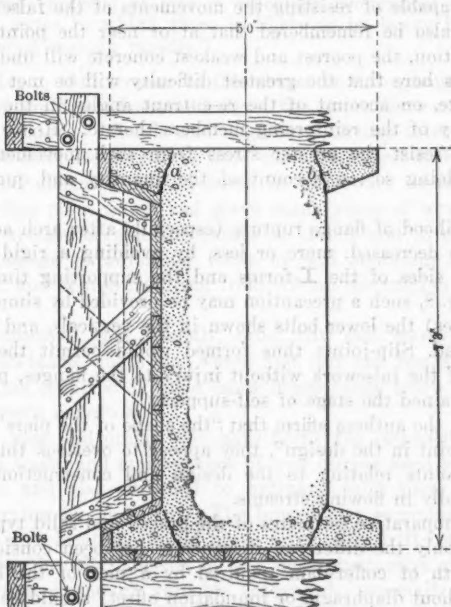
The seeming economy of the rib of I-section, as compared with the rectangular, will not show so vividly in practice as on paper. This saving in concrete in the I-section will be largely, if not entirely, offset by the greater expense caused by the additional material and labor involved in making and placing the I-forms and depositing the concrete therein. Forms for such sections also necessitate the cutting up of the lumber into many short lengths, thus making much of the material unfit for other use. For instance, Fig. 8 shows an I-section form designed for the purpose of economizing in material and in the labor of making, placing, and dismantling it. This form is also capable of repeated use, as it is built in sections with this in view. Yet the additional material and labor involved, in comparison with forms of rectangular section, is apparent at first glance.

In addition, the I-section rib invites and makes possible injury and defects to the permanent structure from which the rectangular section is exempt. No one can foresee or forestall settlements in falsework or centering from timber shrinkage, "taking up", "biting", columnar shortening, foundation movements, expansion, contraction, etc., etc. Falsework or centering moves upward as well as downward on account of the unsymmetrical placing of the load, or in cases where the falsework becomes wet by rain or flood after being thoroughly dried out. A rectangular arch rib is capable of sustaining more settlement or upward movement of the falsework without injury than is possible with an I-section, because of the projecting flanges of the latter. Settlements or movements of the falsework prior to the removal of the I-forms seriously endanger these concrete flanges, and will probably cause rupture at or near their junction with the body or web of the section, as at *a, b, c,* and *d,* of Fig. 8.

From a number of cases, it has been observed that the maximum movements of falsework (both up and down) occur during or imme-

Mr. Aylett.

diately after the construction of the arch ribs (except in cases of accidents, etc.); because this is the period of the initial adaptation or accommodation of the falsework to its superimposed load. During this period, therefore, the I-forms are always found in position and cannot be dispensed with. The danger of flange rupture is most imminent during the initial stages of arch action; that is, when arch action begins to assert itself, the concrete being comparatively green.



Note:
Forms Built in 6 to 10-Ft. Longitudinal Sections
Each Section Detached by Removal of Bolts in Verticals

Fig. 8.

Flange rupture may occur, however, after the rib has passed into the self-supporting stage, or even before arch action asserts itself at all. For instance, if a downward movement of the falsework occurs just as arch action begins to take place, the rib offers some resistance to the movement, though it may be incapable of entire self-support. The result is rupture in the bottom flanges and cracks opening upward, as at *d*, Fig. 8.

Mr. Aylett. If an upward movement of the falsework occurs (on account of swelling, distortion, or accumulation of drift and débris against the forms), flange rupture takes place in the opposite direction, the cracks opening downward, and both top and bottom flanges may be affected, as at *a*, *b*, and *c*, Fig. 8. Flange rupture prior to arch action is most likely to occur in connection with the use of heavy reinforcing members, such as I-beams, lattice girders, etc., as these, by their own arch action, are capable of resisting the movements of the falsework.

It must also be remembered that at or near the points, *c* and *d*, in the I-section, the poorest and weakest concrete will undoubtedly be found. It is here that the greatest difficulty will be met in securing good concrete, on account of the re-entrant angles in the forms, and the proximity of the reinforcing members thereto. Hence the flanges which must resist the greater stress from such movements are less capable of doing so, on account of the quantity and quality of the material.

The likelihood of flange rupture (especially after arch action in the rib) may be decreased, more or less, by avoiding a rigid connection between the sides of the I-forms and the supporting timbers below. Thus, in Fig. 8, such a precaution may be provided by simply omitting (on both sides) the lower bolts shown in the verticals, and using stop-blocks instead. Slip-joints thus formed would permit the downward movement of the falsework without injury to the flanges, provided the arch had attained the stage of self-support.

Although the authors affirm that "the shape of the piers" is a "most important point in the design", they appear to overlook this and other important points relating to the design and construction of bridge piers, especially in flowing streams.

In the comparative estimates of the braced and solid types of piers, apparently, only the difference in concrete has been considered. The greater length of coffer-dam required by a pier of the braced type (with or without diaphragm or foundation offset) should be considered. In constructing coffer-dams in flowing streams, the important thing is usually time, for more risk is attached to coffer-dam building in flowing streams than in any other operation connected with ordinary bridge building, especially where there is "ice or logs of wood moving rapidly".

The many-sided dam of the braced type pier requires much maneuvering of the pile-driver in order to effect the frequent changes of direction. This involves change of anchorage, backing, going forward, etc., etc., all by men who are not sailors, and while breasting a current, and with risk of disaster by flood.

The solid rectangular pier requires less coffer-dam, less maneuvering of the pile-driver, less exposure to flood at the most critical time, and, consequently, much less time and expense. These are among the im-

portant points which should be considered in pier design and construction, and cannot be overlooked in any estimates. Mr. Aylett.

There are, however, other features of the authors' braced pier which appear so serious as to preclude entirely this type from consideration in streams of the character mentioned.

The authors state:

"This type [braced] of pier is especially economical for a bridge in running water in which there is likely to be heavy blocks of ice or logs of wood moving rapidly."

Even prior to 1913 streams of this character throughout the country indicated, by ample evidence and signs, the imprudence of obstructing their flow. Railway officials and engineers, having interest in many streams over extended territory, were among the first to be impressed. Frequent and alarming flood records in numerous streams were observed. Bridge piers which had given many years of service, without trouble or expense, became the causes of claims and litigation on account of damage to adjoining lands by soil stripping and deposits of mud, silt, and débris, resulting from stream obstruction and current deflection.

It was observed that, usually, mud, silt, etc., were deposited in greatest abundance where the water first left the banks of running streams. This occurred in most cases immediately below bridge piers or other current deflectors.

Railway men and others who have borne the responsibility (night and day) of traffic and maintenance of way during flood periods, know the expense and risk attached to ordinary solid piers in running streams carrying ice and drift. The braced pier recommended by the authors for such locations would prove a veritable drift-trap and, viewing the matter broadly, appears to be a most obstructive and expensive type. Cut-waters, though valuable safeguards, especially against ice, do not prevent the lodgment of all logs and drift, even when solid piers of comparatively less widths are used.

The authors' braced type of pier is exposed, not only to the accumulation of drift, etc., on its nose or up-stream end, as in solid piers, but will also catch and hold rigidly logs and drift in the space between the legs on both sides (Fig. 9). Drift thus trapped and held would not float and rise with the flood as it does with solid piers; and, as it would be quickly submerged when caught, it would be almost impossible for an ordinary section gang, even with machinery, to dislodge it.

The buoyancy and enormous leverage of a mass of logs and drift, thus trapped, would exert a simultaneous lifting and wrenching action on the pier, increasing with the pressure of the rising flood on the growing mass held firmly before it.

Mr. Aylett. Drift which collects at the noses of solid piers can always be removed by ice-hooks or pike-poles and floated away. Masses of drift cause deep scour around piers, and this would probably extend below the diaphragm, if its depth were only "down to where they [the pier-legs] enter the ground." If this should occur, heavy drift from both sides might find its way under the diaphragm as well as the braces. It is difficult to reconcile the obstructive braced pier recommended by the authors with the nation-wide conservation movement of to-day, especially in the face of the experience of the West and Middle West in 1913, with its lessons so clearly, impressively, and profoundly taught.

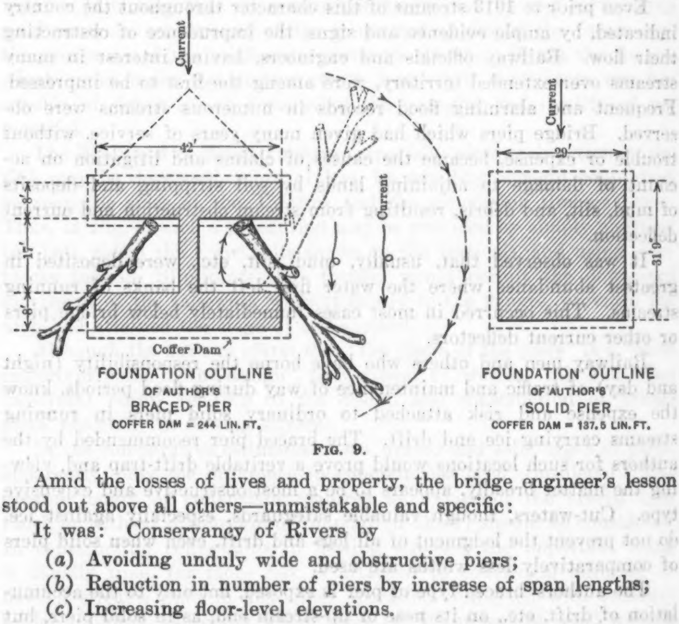


FIG. 9.

Amid the losses of lives and property, the bridge engineer's lesson stood out above all others—unmistakable and specific:

It was: Conservancy of Rivers by

(*a*) Avoiding unduly wide and obstructive piers;

(*b*) Reduction in number of piers by increase of span lengths;

(*c*) Increasing floor-level elevations.

These would facilitate the discharge and consequently help to lower the flood level. By so doing, other lives and property may be saved, and the sacrifices in the West and Middle West will not have been in vain.

Mr. Mensch. L. J. MENSCH, M. AM. SOC. C. E. (by letter).—The statement of the authors that three-hinged arches contain only about half as much masonry as properly designed fixed arches is only true for bridges of very small rise, and the writer would be interested to see the calculations for the bridges shown in Fig. 3, as his own tables do not

show such a difference. The fact is that the thrust in both types is practically the same. The bending moment caused by a concentrated load in a three-hinged arch is greatest when the load is 0.211 l from the abutment, and its value is about $P \times \frac{l}{10}$; in a fixed arch a concentrated load produces the greatest bending moment when placed 0.3 l from the abutment, and its value is about $P \times \frac{l}{16}$. Mr. Mensch.

It is customary to calculate a bridge for a uniform load, w, placed at its half span. The greatest bending produced in a three-hinged arch occurs at the quarter span, and amounts to $\frac{w \times l^2}{64}$; in a fixed arch it occurs at the point 0.3 l from the abutment, and amounts to $\frac{w\,l^2}{120}$; at the same time the moment at the abutment is $\frac{w\,l^2}{64}$.

There is nothing to justify the statement that there is a considerable saving in materials in a three-hinged arch over a fixed arch, unless the temperature stresses and the stresses due to the shortening of the arch and the shrinkage of the concrete play an important rôle, which is only the case in arches of comparatively small rise and large depth. Professor Howe, in his treatise on arches, showed that the thrust from a change of temperature in a parabolic arch of constant $\frac{d\,s}{I}$ may be found by the formula, $\frac{45}{4}\,\frac{E\,I\,c\,t}{f^2}$, when E = the modulus of elasticity, I = the moment of inertia of the crown section, c = coefficient of expansion, t = change of temperature, in degrees Fahrenheit, and f = rise of arch; and that its point of application should be assumed at $\frac{1}{3}\,f$ below the crown. The writer finds that in a parabolic arch, as usually designed, the thickness of the arch increasing at the abutment to about $1\frac{1}{2}$ to 2 times the crown thickness, the thrust may be found by the formula, $24\,\frac{E\,I\,c\,t}{f^2}$, and its point of application may be assumed to be $\frac{7}{30}\,f$ below the crown. Let $E = 2\,000\,000$, $t = 50°$, $c = 0.0000055$, or $E\,c\,t = 550$.

From a change of temperature, the bending moment is found to be,

at the crown, $$\frac{24\,I}{f^2} \times 550 \times \frac{7}{30}\,f = 3\,080\,\frac{I}{f};$$

at the abutment, $$\frac{24\,I}{f^2} \times 550 \times \frac{23}{30}\,f = 10\,120\,\frac{I}{f}.$$

For a width of 1 ft. of the arch ring, $I = \frac{12\,d^3}{12}$, when d = the thick-

Mr. Mensch.

ness at the crown; the moment at the crown $= 3\,080\ \frac{d^3}{f}$, and the moment at the abutment $= 10\,120\ \frac{d^3}{f}$.

If we consider that the moment of resistance of the crown section for a width of 12 in. (neglecting the reinforcement) $= \frac{12 \times d^2}{6} = 2\,d^2$, and that the moment of resistance at the abutment, when the thickness is twice the crown section $= \frac{12 \times 4d^2}{6} = 8d^2$, we can obtain for the stresses in the extreme fibers caused by the change of temperature, by dividing the moments by the section modulus, the simple expressions, $1\,540\ \frac{d}{f}$ for the crown section, and $1\,265\ \frac{d}{f}$ at the abutment, whatever the span of the arch.

The writer will endeavor to compare the two bridges shown in Fig. 3, as far as the few measurements given will allow. He will assume that the thrust per linear foot may be found by the formula, $\frac{850 \times 128.5^2}{8 \times 25} = 70\,000$ lb. The live load of 100 lb. per sq. ft. distributed over a width of 29 ft. will amount to $\frac{100 \times 29}{17} = 170$ lb. if distributed over the two arch rings 17 ft. wide; and the bending moment in a three-hinged arch at the quarter span

$$= \frac{170 \times 128.5^2}{64} = 44\,000 \text{ ft-lb. per lin. ft.}$$

The crown thickness of the arch is 22 in., and, assuming that the thickness at the quarter point is 33 in., the stresses at the quarter point may be found as follows:

The compressive stress $= \frac{70\,000}{33 \times 12} = 177$ lb. per sq. in.

The stresses in the extreme fibers from the bending moment

$$= \frac{44\,000 \times 12}{12\ \frac{33 \times 33}{6}} = 242 \text{ lb.}$$

or a maximum compressive stress of $177 + 242 = 419$ lb. and a maximum tension of $242 - 177 = 65$ lb.

The average thickness of the arch is $22 + \left(\frac{2}{3} \times 11\right) = 29.33$ in.

In a fixed arch it will be necessary to assume a greater thickness of the arch at the crown, on account of the temperature stresses; therefore, the thickness at the crown will be assumed to be 28 in. and that

Mr. Mensch.

at the abutment, 56 in. The maximum bending moment caused by a uniform load, w, per square foot at the crown is about $\frac{w \times l^2}{320}$, and the compressive stress from the thrust $= \frac{70\ 000}{12 \times 28} = 208$ lb. per sq. in.; the stresses in the extreme fibers, from the uniform load, $= \frac{170 \times 128.5^2}{320}$, divided by $\frac{12 \times 28^2}{6} = 56$ lb. per sq. in.

As shown before, the temperature stress in the extreme fibers $= 1\ 540 \frac{d}{f} = 144$ lb. per sq. in., or a maximum compressive stress of $208 + 56 + 144 = 408$ lb. and a minimum compressive stress of $208 - 200 = 8$ lb. per sq. in. At the abutment, the thrust is $70\ 000 \times 1.28 = 89\ 500$ lb. and the compressive stress $= \frac{89\ 500}{12 \times 56} = 133$ lb. The maximum bending moment from a uniform load

$$= \frac{170 \times 128.5^2}{64} = 44\ 000 \text{ ft-lb.}$$

The section modulus at the abutment $= \frac{12 \times 56^2}{6} = 6\ 280$ in.^{3}, and the stresses in the extreme fibers $= \frac{44\ 000 \times 12}{6\ 280} = 84$ lb.

The temperature stresses $= 1\ 265 \frac{d}{f} = 118$ lb., or the maximum compressive stress $= 133 + 84 + 118 = 335$ lb. per sq. in., and the maximum tensile stress $= 202 - 133 = 69$ lb. per sq. in. The average thickness of the arch $= 28 + \left(\frac{1}{3} \times 28\right) = 37.33$ in.

The stresses from the shortening of the arches will modify the results only very slightly in this case. Although both bridges have about the same stresses, the average quantity of concrete is only 27% greater in the fixed than in the three-hinged arch. Or, assuming that 200 cu. yd. for the three-hinged arch is correct, the saving amounts to only 54 cu. yd., or $675, which is more than counterbalanced by the cost of steel hinges.

The authors state that the slightest movement of the abutments will develop cracks in the arches. This is not the case. A temperature drop of 50° is equivalent to a movement of both abutments of $\frac{1}{3\ 640}$ of the span, or, in this case, 0.42 in., and it is a poorly designed abutment in which this movement will be seen. Such a movement would increase the stresses at the crown to $408 + 144 = 552$ lb. in compression and $144 - 8 = 136$ lb. in tension, and at the abutment to 335 +

Mr. Mensch. 118 = 453 lb. in compression and 69 + 118 = 187 lb. in tension. It is known that ordinary slabs do not crack under stresses less than 300 lb. per sq. in.; hence the factor of safety against the first crack is still ample.

It is true, however, that the temperature stresses and those due to the shortening of the arch would be three or four times as great in the same arch with a rise of only one-tenth of the span, and the advantage of the three-hinged arch in such a case is unquestioned.

In a three-hinged arch, great stresses may be caused by the unequal settlement of the staging, and it has often happened that the hinges have been thrown out of line and out of center, and have had to be dug out and replaced. The failure of a large bridge was due to this cause. This is the main reason that fixed arches are preferred by experienced engineers to the three-hinged type.

It must also be mentioned that three-hinged arches settle considerably more than fixed arches after the centering is struck, and also cause more vibration under moving loads.

In three-hinged arches, the use of deep ribs is a decided saving, which is not the case in all fixed arches. The writer has advocated* the use of I-sections for spans of more than 300 ft., and for spans of from 200 to 300 ft. he advocates T-sections.

The rule given by the authors for finding the center line for a three-hinged arch can be simplified by assuming the entire arch to be loaded by one-half of the uniform load per square foot. This, according to the elastic theory, gives the most favorable line of pressure.

Mr. Buel. A. W. Buel, M. Am. Soc. C. E. (by letter).—Mr. Gregory, Mr. Janni, and others have brought out most of the points involved, but there remains one of vital importance as affecting the comparison of the hinged and fixed-arch rib as made by the authors. Although they have used the proper form for their hinged ring, their fixed ring is neither correct nor economical, and, therefore, is not a fair comparison. Being segmental, it is subject to great moments at the springing line, as the ring does not conform to any pressure line for any practical conditions.

The only economic justifications for the three-hinged ring are difficult foundation conditions and a rise of less than one-sixth of the span with a possible yielding of the abutments.

The authors exaggerate the difficulty of computing the fixed arch, for, by systematizing and tabulating the work, it is quite possible to make the computations in 2 days or less. Moreover, it does not seem to be good engineering practice, nor creditable to the industry of the Profession, to advocate a design because it may save a few hours or even a few days of work for the designer. This is aggravated when there is room for the suspicion that any such saving in the designer's

* "The Reinforced Concrete Pocket Book."

time and energy is largely due to his unfamiliarity with fixed-arch computations. Mr. Buel.

The advantages of the ribbed ring have been demonstrated and advocated* by the writer in designs worked out in 1899 and 1901.

W. D. Maxwell, Assoc. M. Am. Soc. C. E. (by letter).—The question of temperature changes in arches has been mentioned by several who have discussed this paper. Mr. Maxwell.

When the Walnut Street Concrete Bridge, of which the writer was one of the designers, was built in Des Moines, Iowa, in 1911, nine electrical resistance thermometers were embedded at various points in one of the arches. This arch has a clear span of 68 ft., a crown thickness of 16 in., and a solid spandrel fill. The greatest range in temperature was at the crown, where an average of two thermometers showed a temperature of 86° Fahr. about 16 hours after pouring, 9° below zero as the low point during the following winter, and 86° in the summer of 1912, one year after the bridge was built. Other thermometers placed in thicker portions of the arch showed somewhat less susceptibility to atmospheric changes. The average of two instruments embedded about 2 ft. in heavy concrete near the pier and abutment showed 98° Fahr. about 36 hours after pouring, 20° the next winter, and 85° the following summer. The average of all the thermometers for the first 6 months, gave a range of temperature in the concrete of 87.6 degrees. The upper limit of this range was the extreme produced by the heating of the setting concrete. The average of all the thermometers for the second 6 months was 79.2 degrees.

The extremes of atmospheric temperature in the summer of 1911 and the following winter were very great for this locality, the total range being 135 degrees. However, ranges of considerably more than 100° may be expected every year.

Elevations were taken on all six arches of the bridge in September, 1911, when the average of all the thermometers was 75°, and again in January, 1912, when the average was 11 degrees. The difference corresponded closely to that computed for a change of 64° in the temperature of the concrete.

In the design of the bridge, allowance was made for a 40° rise, and a 40° fall. These amounts were considered excessive by some engineers, but the actual observations show that they were not only not too small, but that the fall was more than 40°, the concrete having been poured at a general average of about 70 degrees.

Considering these temperature ranges, it is apparent that concrete bridges built in these rigorous climates, especially the hingeless arches, must be very carefully reinforced against temperature stresses, and it is also evident that the warmer climates and those having nearly uni-

* "Reinforced Concrete," First Edition, 1904.

Mr. Maxwell. form yearly temperatures possess a real advantage with this type of construction.

Mr. W. M. Smith, Sr. WALTER M. SMITH, SR., M. AM. SOC. C. E. (by letter).—Mr. Gregory states that the decision to adopt a three-hinged arch for the Traver Hollow Bridge was taken because the foundations for the abutments and piers would be on compressible material. He also states that the preliminary estimates indicated that the three-hinged arch would be somewhat more expensive than the fixed-arch type. He is correct in the first statement, but is mistaken in the second. Only a rough estimate was made for a fixed arch at this site, making certain assumptions as to the depth to rock. Careful designs and estimates were then made for a three-hinged arch, to compare with the other. The writer had charge of both estimates, and made the design and estimate for the three-hinged arch. He has copies of these estimates before him as he writes. If the same unit prices are taken for the three-hinged arch as for the fixed arch, the saving would be about 30% for the former. Therefore the average increase in unit price for the three-hinged arch would have to be 43% in order that it might cost as much as the fixed arch for this site. When these estimates were obtained, as there was no information at hand as to the depth to rock, it was decided to build the three-hinged arch, as it was certain that the material would carry the load of an arch of this type by compressing slightly, and would not injure the arch.

Figs. 10, 11, and 12 show the details of the hinges and the method of connecting them to the steel ribs of the arch. It will be seen that the steel forms a continuous structure between the abutment hinges, with an additional hinge at the crown.

When the design of the Rye Outlet Bridge was under consideration, the writer also investigated a three-hinged arch for this site, at Mr. Gregory's request. As the height of the roadway was fixed, and also the water line, the design was limited to short arches, unless it was admissible to construct them with a ratio of rise to span of more than six, on very high piers. It was thought inadvisable to do this. With short arches the three-hinged type showed such a slight saving over the fixed type that the latter was adopted, as the piers were to be founded on bed-rock.

Mr. Gregory is wrong in supposing that the pier shown in the paper is one of those in the Rye Outlet Bridge. In reference to this pier, the paper states that, "The first is a braced pier somewhat similar to those in the Rye Outlet Bridge." The thrusts, shown as acting at the top of this pier, are assumed, and, so far as the writer knows, they may not be anywhere near those of the piers in the Rye Outlet Bridge. They were taken arbitrarily, and the piers were designed to suit the thrusts; therefore the writer fails to see how the investigation mentioned by

Mr. W. M. Smith, Sr.

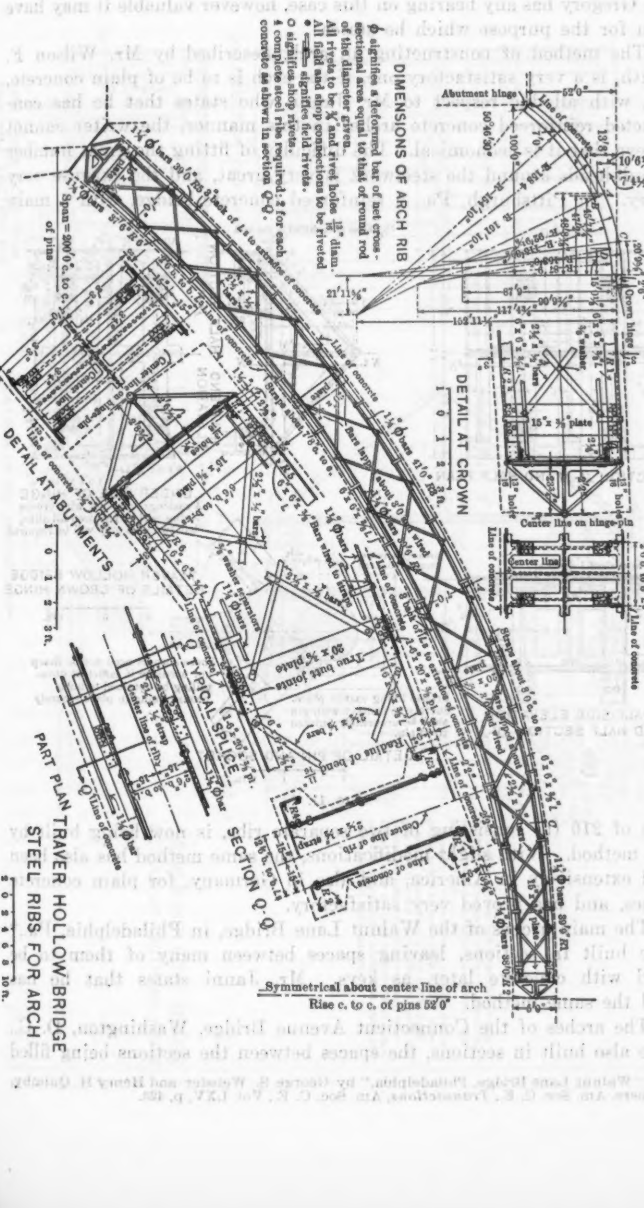


Fig. 10.

Mr. W. M. Smith, Sr.

Mr. Gregory has any bearing on this case, however valuable it may have been for the purpose which he states.

The method of constructing an arch, described by Mr. Wilson F. Smith, is a very satisfactory one, if the arch is to be of plain concrete, but, with all due respect to Mr. Janni, who states that he has constructed reinforced concrete arches in this manner, the writer cannot believe that it is economical. The difficulty of fitting the great number of bulkheads around the steelwork is very great, and the expense very heavy. In Pittsburgh, Pa., a reinforced concrete bridge, with a main

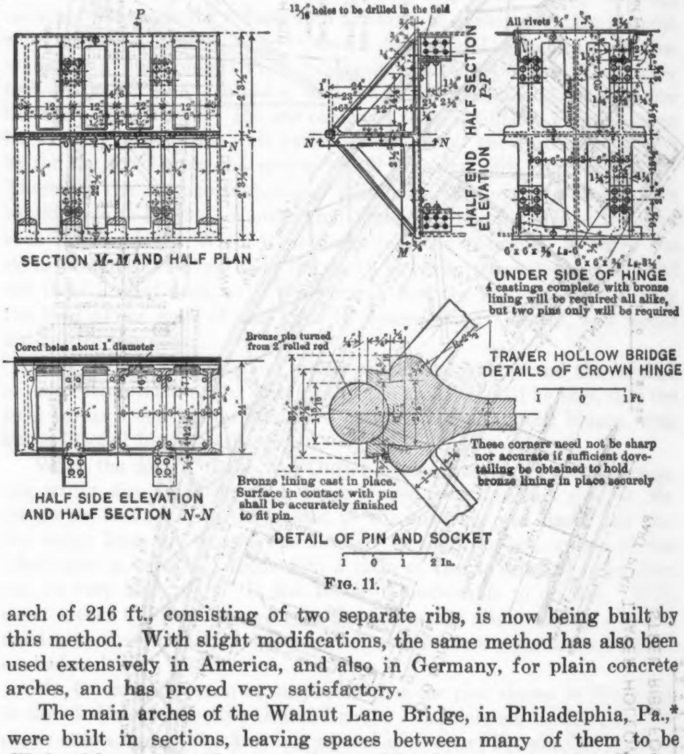


Fig. 11.

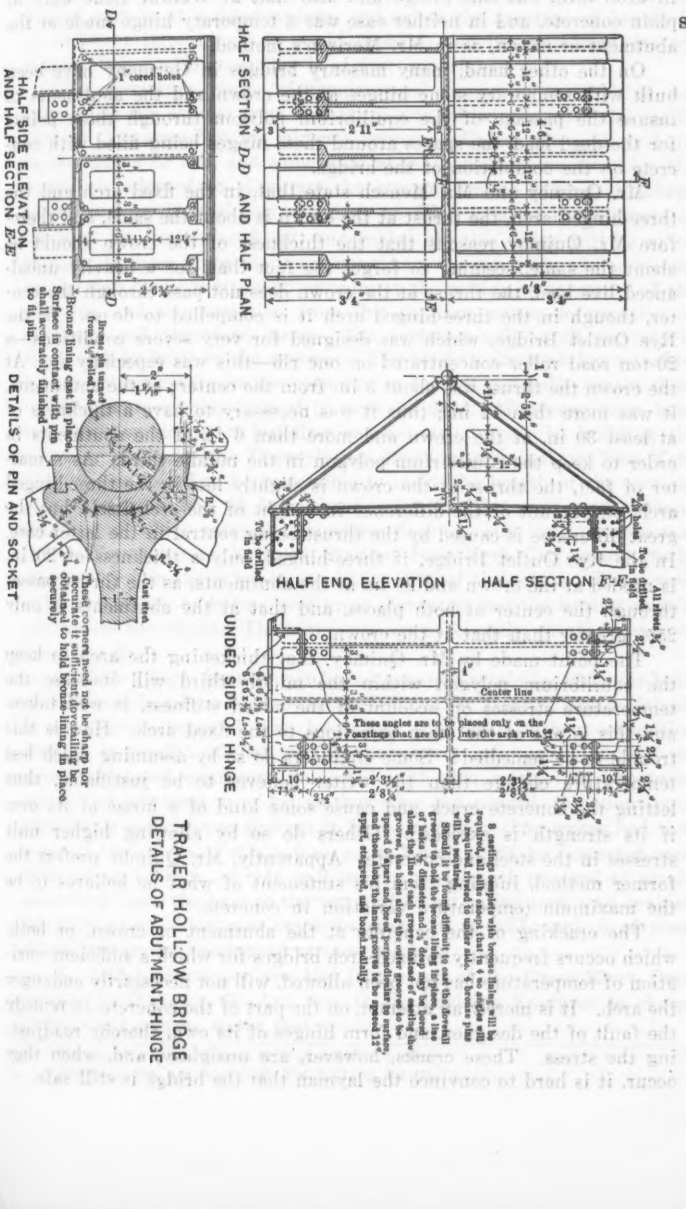
arch of 216 ft., consisting of two separate ribs, is now being built by this method. With slight modifications, the same method has also been used extensively in America, and also in Germany, for plain concrete arches, and has proved very satisfactory.

The main arches of the Walnut Lane Bridge, in Philadelphia, Pa.,* were built in sections, leaving spaces between many of them to be filled with concrete later, as keys. Mr. Janni states that he has used the same method.

The arches of the Connecticut Avenue Bridge, Washington, D. C., were also built in sections, the spaces between the sections being filled

* "Walnut Lane Bridge, Philadelphia," by George S. Webster and Henry H. Quimby, Members. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXV, p. 423.

Mr. W. M. Smith, Sr.



TRAVER HOLLOW BRIDGE
DETAILS OF ABUTMENT HINGE

FIG. 12.

Mr. W. M. Smith, Sr. in afterward, but this bridge and also that at Walnut Lane were of plain concrete, and in neither case was a temporary hinge made at the abutment or crown, as in Mr. Morison's method.

On the other hand, many masonry bridges in Germany have been built with temporary stone hinges at the crown and the abutments to insure the passage of the equilibrium polygon through these points for the dead load, the spaces around these hinges being filled with concrete on the completion of the bridge.

Mr. Quimby and Mr. Mensch state that, in the fixed arch and the three-hinged arch, the thrust at the crown is about the same, and therefore Mr. Quimby reasons that the thickness of the crown should be about the same, seeming to forget the fact that, for a heavily unbalanced live load, the thrust at the crown does not pass through the center, though in the three-hinged arch it is compelled to do so. In the Rye Outlet Bridge, which was designed for very severe conditions—a 20-ton road roller concentrated on one rib—this was especially so. At the crown the thrust was about 5 in. from the center; at the abutments, it was more than 12 in.; thus it was necessary to have a thickness of at least 30 in. at the crown and more than 6 ft. at the abutments in order to keep the equilibrium polygon in the middle third. As a matter of fact, the thrust at the crown is slightly less in the three-hinged arch, on account of the difference in weight of the arch itself, but the great difference is caused by the thrust being central in the latter case. In the Rye Outlet Bridge, if three-hinged, only a thickness of 20 in. is needed at the crown and 25 in. at the abutments, as the thrust passes through the center at both places, and that at the abutments is only 25% greater than that at the crown.

The point made by Mr. Quimby, that thickening the arch to keep the equilibrium polygon within the middle third will increase the temperature stresses on account of the added stiffness, is well taken, and this is one of the main objections to the fixed arch. How is this trouble to be remedied? Some engineers do so by assuming much less temperature change than the writer believes to be justifiable, thus letting the concrete crack and cause some kind of a hinge of its own if its strength is exceeded. Others do so by allowing higher unit stresses in the steel and concrete. Apparently, Mr. Quimby prefers the former method, judging from his statement of what he believes to be the maximum temperature variation in concrete.

The cracking of the concrete at the abutment or crown, or both, which occurs frequently in fixed-arch bridges for which a sufficient variation of temperature has not been allowed, will not necessarily endanger the arch. It is merely an attempt, on the part of the concrete, to remedy the fault of the designer, and form hinges of its own, thereby readjusting the stress. These cracks, however, are unsightly, and, when they occur, it is hard to convince the layman that the bridge is still safe.

The writer has stated that, with regard to appearance, the three-hinged arch is at a disadvantage, and, when the question of beauty is of sufficient importance, it would probably not be considered, its advantage being where economy is of first importance. Mr. W. M. Smith, Sr.

The maximum range of temperature change in an arch is not necessarily the change from midsummer to midwinter, with the condition of "no stress" lying midway between. Unless the arch is built in the spring or fall, and some method (such as that described by Mr. Janni) is adopted to eliminate the stresses due to setting shrinkage by leaving in the arch key spaces to be filled in after the concrete of the arch has been placed, set, and cooled off, this range will be much higher. If these precautions are not taken, the condition of "no stress" in the arch is the position it occupies at the setting temperature of the concrete, and any variation from this position causes deformation of the arch, and, therefore, stress. The variation will be all one way if the bridge be built in the summer. The steel in the arch has no effect in preventing cracks in the concrete, in fact, it generally causes them to occur more quickly than if it were not used. Many engineers have had this experience in building various structures. In building the retaining walls for the Pennsylvania Railroad terminal, in New York City, it was found to be impossible to make the length of the sections of wall greater than 25 ft. without cracks.* The reinforced concrete floor beams, which carry the concourse floor of vault light construction in the Pennsylvania Railroad Station, New York City, show cracks where they join the columns at almost 50% of the points. These were built as continuous beams over great distances, and the setting shrinkage caused the cracks. The beams are not necessarily unsafe because of these cracks.

In 1908 a high, heavily reinforced concrete retaining wall was built along the railroad yard on the water-front at St. George, Staten Island. It is several thousand feet long, and was built without expansion joints. In the fall of that year the writer was informed, by one of the most prominent engineers connected with this work, that expansion joints had not been provided as they were not needed, the quantity of steel being sufficient to prevent it from cracking. The writer had seen some of the effects of this before, so in March, 1909, he inspected carefully about 1 000 ft. of this wall, and in that length counted 45 cracks extending from the base to the top, and 17 extending part way, making an average of about one crack in every 16 ft. Through 8 of these cracks water was issuing at various heights.

A similar experience was had in the top of the Cross River Dam. George G. Honness, M. Am. Soc. C. E., Division Engineer in charge of its construction, states that:†

* *Transactions*, Am. Soc. C. E., Vol. LXVIII. p. 334.

† *Transactions*, Am. Soc. C. E., Vol. LXI. pp. 411-418.

Mr. W. M. Smith, Sr.

"It was not expected that the use of these rods would entirely eliminate the temperature cracks, but it was hoped that it would prevent their concentration in a few large cracks.

* * * * * * *

"The reinforcement * * *—as to quantity and method of placing—was not effective in preventing the concentration of the cracks."

The following is given as an example of how greatly concrete shrinks in setting. On some work with which the writer was connected, some years ago, it became necessary to cut out some concrete in a ceiling consisting of a very deep and heavy concrete slab with reinforcing bars embedded about 2 or 3 in. from the lower surface. The concrete was cut from around a few of these bars for a distance of about 2 ft. All the bars, when released, sprang out of line, thus showing that the shrinkage of the concrete, in that short distance, was sufficient to shorten the steel bars by compressing them. Theoretically, these bars were carrying a tension of 16 000 lb. per sq. in. In reality, they were in compression, and the concrete was not only carrying all the load, but was also compressing the steel.

Many experiments have been made in order to determine the rise in temperature in cement mortar, neat and of various compositions, and of concrete during setting. In some tests by the Universal Portland Cement Company on neat cement cubes, the average temperature rise was from 80° to a little more than 130° Fahr., in from 6 to 8 hours after placing. In the case of concrete, the initial temperature, 70° Fahr., was increased to 80° in 8 hours, 100° in 25 hours, and 105° in 45 hours. Tests in the laboratory of Lehigh University have shown that the temperature of an 8-in. cube of 1 : 3 Portland cement mortar rose from an initial temperature of 70° to 160° Fahr. in 18 hours. At the Watertown Arsenal, a few years ago, tests of 29 cubes of neat cement, representing 27 brands of cement, were made. The initial temperature was 77°, and the average maximum 185° Fahr., showing a rise of 108° in an average of 13 hours. The thermophones embedded in the cyclopean masonry of the Boonton Dam indicated a rise from 20 to 25° Fahr., although only about two-thirds of the mass consisted of concrete. In the locks of the Panama Canal the rise was also about the same.

Table 1 shows the setting temperature in some concrete canal walls at Madison, Me., built for the Hollingsworth and Whitney Company. As the concrete was placed in the winter, it was first heated to 70° Fahr. The average atmospheric temperature was 11° and that of the concrete 108°, showing a rise of 38° after being placed, in spite of the exceedingly low outside temperature.

As an example of the low temperature which may be reached in bridge arches in the Northern States, Table 2, showing the results of tests on the Squaw Creek Bridge, at Ames, Iowa, is interesting.

TABLE 1.—SETTING TEMPERATURE IN CONCRETE CANAL WALLS AT MADISON, ME.

Mr. W. M. Smith, Sr.

Date. 1909.	Depth of concrete, in inches.	Atmospheric temperature, in degrees, Fahrenheit.	Temperature of concrete, in degrees, Fahrenheit.
Jan. 19th	3.0	— 18	91
20th	3.0	— 2	94
21st	7.5	14	96
22d	15.5	34	98
23d	15.5	39	106
24th	15.5	20	110
25th	15.5	15	111
26th	15.5	18	113
27th	26.25	2	119
28th	35.0	18	119
29th	35.0	14	116
30th	37.0	21	116
31st	37.0	20	113
Feb. 1st	37.0	0	113
2d	37.0	— 5	110
3d	37.0	— 10	109
4th	37.0	— 11	113
5th	37.0	25	114
6th	37.0	28	106
7th	37.0	17	104
8th	37.0	6	101
9th	37.0	— 4	98
Averages		11	108

Mr. Maxwell has given a brief account of the range of temperature experienced in the Walnut Street Bridge, Des Moines, Iowa, and readers are referred to a synopsis of his original report on this bridge, and also on some others.* Many of the data herewith presented have been obtained from Mr. Maxwell's valuable report. Tables 3 and 4, taken from that report, are interesting.

TABLE 2.—RESULTS OF TESTS ON SQUAW CREEK BRIDGE, AMES, IOWA.

Thermometer.	Maximum.	Minimum.	Range.
A1	82.2°	— 2.5°	84.7°
B1	79.5°	2.0°	77.5°
A2	80.5°	8.0°	72.5°
B2	81.4°	— 0.5°	81.9°
Average range			79.1°

The coils were in different portions of the structure, and at different distances from the surface. Coils Nos. 2 to 5 were placed in the concrete where the arch joins the abutments or piers. No. 1 in the

* *Engineering and Contracting*, November 12th, 1913.

Mr. W. M. Smith, Sr.

TABLE 3.—TOTAL TEMPERATURE RANGE FOR THE WALNUT STREET BRIDGE.

Coil No.	Minimum temperature, in degrees, Fahrenheit.	Maximum temperature, in degrees, Fahrenheit.	Range of temperature, in degrees, Fahrenheit.
PERIOD FROM AUGUST, 1911, TO JANUARY, 1912.			
2	15	108	93
3	20	102	82
4	19	95	76
5	18	96	78
6	0	88	88
7	— 5	90	95
8	— 9	86	95
9	— 9	85	94
Average range			87.6
PERIOD FROM JANUARY TO AUGUST, 1912.			
2	15	84	69
3	20	83	63
4	19	86	67
5	18	85	67
6	0	87	87
7	— 5	86	91
8	— 9	84	93
9	— 9	88	97
Average range			79.3

TABLE 4.—RISE IN TEMPERATURE DUE TO SETTING.

Coil No.	Initial temperature of concrete, in degrees, Fahrenheit.	Maximum temperature of concrete, in degrees, Fahrenheit.	No. of hours after placing.	Rise of temperature, in degrees, Fahrenheit.
1	80	107	75	27
2	76	108	43.5	32
3	76	102	27.5	26
4	80	89	75	15
5	76	96	18	20
6	76	88	10	12
7	78	90	23	12
8	76	86	10	10
9	78	84	23	6
Averages	77.4	94.5	33.6	17.1

center of the pier, Nos. 6 and 7 in the center of the arch half way between the springing line and the crown, and Nos. 8 and 9 in the center of the arch at the crown. The arch is not exposed on its upper side, as there is an earth fill above it. Coils Nos. 6 and 7 are about 9½ in. from the surface, and Nos. 8 and 9 about 8 in., consequently the heat

generated by the setting would be dissipated much more rapidly at Nos. 6, 7, 8, and 9, than at the others. Conversely, these four reach temperatures much lower in winter and somewhat higher in summer than the others. As the upper surface of the arch is protected by an earth fill, the range there is probably not as great as it would be if the arch had open spandrels, therefore the variations are on the conservative side. Mr. W. M. Smith, Sr.

It is evident, in this case, that the concrete set and the arch took its shape at a temperature of more than 90 degrees. Any deviation from this temperature, then, would cause stress in the arch. As all the thermometers in the arch itself reached zero, or went below zero, during the following winter, the arch was subjected to a stress due to a variation of more than 90° from the normal. The arch was designed for a variation of 40° each way from the normal, therefore it was subjected to 2¼ times as great a range as that for which it was designed. What is likely to happen in such a case? Probably the arch will crack, either at or near the abutments, or at the crown, or at both. As the arch at the abutments or piers is much deeper, and therefore much more rigid, than at the crown, the cracks are more likely to appear there. When the concrete cracks, the stress is readjusted, and the arch has formed somewhat of a hinge for itself. As stated previously, the arch is not necessarily endangered by these cracks, as they cause a readjustment of the stress, but they are unsightly, and it is difficult to convince a layman that the arch is perfectly safe.

In his report Mr. Maxwell states that at that time there were no cracks in the Walnut Street Bridge, but in the Squaw Creek Bridge there are unsightly cracks at the exact point of computed maximum stress at three of the four corners of the bridge. He also states that in the Locust Street Bridge, Des Moines, Iowa, there are cracks at the crown of each of the shore spans.

As all these arches were designed for a variation of 40° each way from the normal, Mr. Maxwell concludes that for northern latitudes a variation of this much should be allowed, and that:

"Particular circumstances may demand that a greater variation be used for drop in temperature to prevent the appearance of cracks."

This conclusion was reached by the writer some years ago. If, in the face of these facts, an engineer is willing to design a bridge for a variation of only 20° each way from normal, as Mr. Quimby seems to propose, and then lets the bridge crack and form its own hinges, the writer freely admits that it will be a cheaper bridge than a three-hinged arch designed for reasonable stresses.

It is admitted that, if a stone hinge be used in an arch, the friction of the hinges should be taken into consideration in computing the stress due to change of temperature, but with a metal hinge, such as

Mr. W. M. Smith, Sr. that in the Traver Hollow Bridge, the amount would be so small as to be negligible. The writer would like to call Mr. Janni's attention to the details of these hinges and the arch rib, Figs. 10, 11, and 12. He will see that the span of the arch is 200 ft., and that the diameter of the pin at the abutment hinge is about 3 in., and at the crown 2 in.; therefore the maximum deviation of his lines, P_1 and P_4, would be 10 in. in 110 ft., which is certainly not very serious, to say the least. The pressure on these pins is about 12 000 lb. per sq. in. The socket for holding the pin is very heavy and rigid, so as to cause very nearly an even distribution of pressure on the pin, and a lining is cast in the socket and then finished to fit the pin accurately. The metal of the body of the hinge is cast steel having an elastic limit of 40 000 and a tensile strength of 76 000 lb. per sq. in., an elongation of 20% in 2 in., and a reduction of area of 41.5 per cent. A sample of this steel, ½ in. thick, was bent 180° around a diameter of 1½ in. without a sign of cracking either on the outside or inside of the bent portion. The socket of the casting was lined with "Monel" metal by casting it in place and then machining it to fit the pin accurately. The pin was of the same metal, finished from a rolled bar. This metal is a natural alloy of approximately the following composition: nickel 68%, copper 31%, and iron 1 per cent. The writer has a sample of this metal which tested as follows: tensile strength, 100 000 lb. and elastic limit 71 600 lb. per sq. in., elongation, 35% in 2 in.; reduction of area, 54.7 per cent. Now, with this metal as a bearing for the hinges, even if there was the concentration of pressure which Mr. Janni thinks is so serious, there would still be a factor of safety of 3 on the elastic limit of the material, and this, the writer thinks, is sufficient.

For the bending moment due to friction on the pin, the coefficient of friction of the pin and lining may be assumed at 0.20. The best authorities give this coefficient, for metal on metal, dry, as from 0.15 to 0.25; and, as this bearing is bronze on bronze, it will probably be nearer the lower limit, so that 0.20 should certainly be conservative. The total pressure on the pin is about 2 000 000 lb., therefore the bending moment of the friction at the surface of the pin, with the lever arm of 1½ in., will be $2\,000\,000 \times 0.20 \times 1.5 = 600\,000$ in-lb., which would be the moment caused by the pressure line deviating from the center line by less than 0.3 in., as the pressure decreases as the distance from the pin increases.

Mr. Janni speaks of taking care of the stress due to dead load and that due to shrinkage by neutralizing their effects, as much as possible, by the method mentioned previously, that is, by building the concrete of the arch in sections and leaving key spaces between them to be filled in after the cement has set—in not less than 10 days, he states. His reference to neutralizing the dead load stresses is supposed to mean the stresses caused by the pressure line for dead load leaving the center

Mr. W. M. Smith, Sr.

line of the arch and thus causing a bending moment, as direct compressive stresses caused by the dead load, of course, cannot be neutralized in any way. All that is necessary to accomplish this is to design the arch so that the center line is the pressure line for the dead loads on the arch. When the dead load has been obtained, it is very easy to do this. As stated previously, this is probably as good a method as can be found, for a fixed arch, to neutralize the stresses due to setting, but is, certainly, very expensive; and it does not affect the stresses due to a change from the atmospheric temperature at which the arch was cast to the extreme variation either way.

In any of the Northern States the average temperature for June, July, and August is higher than 70°, and the winter temperature goes well below zero at frequent intervals. Therefore, in any open spandrel arch built in the Northern States during these three months, the drop in temperature will probably be—as a conservative estimate—at least 60 or 65 degrees. Mr. Maxwell has shown that, in Iowa, at least, the range will be much more than that.

If all arches could be built between March 15th and May 15th, or between September 15th and November 15th, this trouble would not occur, and a method of neutralizing the stresses due to setting, similar to Mr. Janni's and allowing a range of 40° each way from the normal, would probably insure the arch against cracks.

In Mr. Aylett's Fig. 8, an arch of I-section, there would probably be some danger of the flanges breaking off, as he suggests. These flanges should be at least twice as thick as shown by Fig. 8, as it is not practicable to approach the thinness of the steel beam flange with concrete. The proportions used in the Traver Hollow Bridge are shown on Fig. 10.

The writer, however, cannot see that there is much danger of these flanges being broken off, as the load of the arch is carried to the falsework as a distributed load across its lower surface. The paneling of the sides is of light construction and is not independent of the falsework, therefore it has to move with it, and, as the movement of the falsework is caused by a corresponding movement of the arch, the whole structure must, of necessity, move together.

It may be of interest to state that the writer has a letter from the contractor who built the Traver Hollow Bridge, stating that no difficulty was experienced in its construction.

The writer will admit that, if piers are to be built in a flowing stream, in which there are quantities of tree trunks with the boughs all nicely trimmed off a short distance from the trunk, so that they can catch on anything with which they come in contact, as shown in Mr. Aylett's Fig. 9, there might be danger of a quantity of drift collecting around the piers. However, he does not believe that there is much danger of an object, after being deflected by the cut-water,

Mr. W. M. Smith, Sr. being drawn in again quickly enough to be caught by the down-stream leg of the pier. He can see no danger whatever of a tree trunk being deflected by the cut-water and then running in again with its butt between the legs of the pier, and then swinging around and hanging there. The cut-water throws the water away from the pier, and, on account of this obstruction, the water will be slightly higher as it passes, and hence the velocity will be increased. Any object deflected by the cut-water will be carried away from the pier slightly by this elevation of the water, instead of being drawn in.

In streams subject to heavy floods, where any obstruction of the channel would be serious, it is probable that wide piers would be out of the question, and undesirable, no matter what the economy would be in theory. The writer does not wish to be understood as claiming that the braced pier shown should be adopted in all cases, by any means, for he believes that the conditions at every important bridge should be investigated carefully. He thinks, however, that the braced pier is worthy of investigation when the design for a bridge is taken up.

Mr. Mensch is mistaken if he thinks that the writer claims, as a general thing, that the concrete in a three-hinged arch is about 50% less than in one that is fixed, and no statement to that effect was made in the paper. This happened to be a case where the comparison was unusually favorable to the three-hinged arch, as the proportion of rise to span is almost 1 to 6. The writer used this arch because he had designed it some years ago for the Rye Outlet Bridge, and had all the dimensions on hand, and wished to compare a three-hinged arch with it. There are five arches in the Rye Outlet Bridge, and this is the center span taken for a single-arch bridge.

It is an interesting fact that though this comparison is so favorable for the three-hinged arch for a single span, when it is applied to the Rye Outlet Bridge there is very little difference in cost, not more than about 4%, if one does not take into account the steel reinforcement. The reason for this is that the bridge piers are so high and the superstructure so massive that the total cost of the arch concrete is only 13% of the cost of the bridge, and $13 \times 0.48 = 6.25\%$ without allowing for the cost of the hinges. Taking these into consideration, the difference is reduced to about 4 per cent.

In stating the conclusions in the paper the writer did not mean to claim that every individual design would necessarily agree with these conclusions, but that they were true in the average case. He thinks that the paper has brought out a valuable discussion on a subject which, heretofore, has not been treated at length, and is glad that the matter was brought up. He has not found anything in the discussion, however, to cause him to change his conclusions, but has been further strengthened in them.

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Paper No. 1295

STRESSES IN WEDGE-SHAPED REINFORCED CONCRETE BEAMS.

By William Cain, M. Am. Soc. C. E.

With Discussion by Messrs. A. C. Janni, L. J. Mensch, L. H. Nishkian, and William Cain.

In the design of reinforced concrete walls, it is often necessary to find the stresses in beams with faces inclined to each other, as in the toes, heels, face-slabs, and counterforts of such walls. In other cantilever constructions, the upper and lower faces are often inclined to each other, so that the subject is one of importance, and a practical solution is offered in this paper.

First consider the beam, Fig. 1, the upper face of which is inclined at an angle, β, to the lower face, supposed to be horizontal. This beam may be supposed to be the toe of a reinforced concrete retaining wall, and to be subjected to a soil reaction, acting vertically upward, which produces shear and also stresses due to the bending moment, in any vertical section, $N\ I$.

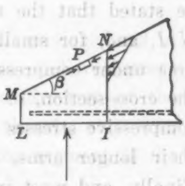

Fig. 1.

To effect a practical solution, it will be assumed that all the compressive bending stresses act parallel to the top face, $M\ N$, down to the neutral axis. This is the direction of the stress at any point of the upper surface, $M\ N$; for, consider a small rectangular parallelopiped of the concrete at P, with faces parallel and perpendicular to $M\ N$. There can be no shearing stress on $M\ N$, as no part of the beam extends above $M\ N$ to produce shear, and there is no external

force acting on $M\ N$ except the atmospheric pressure, which does not exert any friction on the face. It must follow that there can be no shear on the planes at right angles to $M\ N$, as unit shears on planes perpendicular to each other are equal.* Consequently, the compressive stress on a plane at P, at right angles to $M\ N$, is normal to it, or parallel to the face. This conclusion has been proved experimentally by Messrs. Wilson and Gore, in their exhaustive experimental work on india-rubber model dams.† They state, as one conclusion: "The maximum principal stresses near the down-stream face act on planes normal to that face," and add in a foot-note, "This agrees with Rankine's statement and with the theorem demonstrated by Mr. M. Levy." In the discussion on that paper, the same point was brought out by several speakers.

It is then plain that the compressive stress at N, Fig. 1, similarly, acts parallel to the face, $M\ N$. If this were a homogeneous beam, the tensile stress at I would similarly act parallel to the lower face. Consequently, in going from N to I, the intermediate bending stresses would gradually change their direction from $M\ N$ to $L\ I$.‡ Hence the foregoing assumption, that all the compressive stresses are parallel to $M\ N$, is to be regarded only as an approximation necessary to derive workable formulas.

In justification of its use for reinforced beams, however, it may be stated that the neutral axis is generally above the mid-point of $N\ I$, and, for small percentages of steel, much higher; so that the area under compression is often only one-third of the whole area of the cross-section, or even less. Further, the resisting moment of the compressive stresses is mainly due to the larger stresses near N, with their longer arms, and such stresses are nearly parallel to $M\ N$. Finally, and most important of all, the assumption is always on the side of safety, as will appear more fully later. The assumption is evidently near the truth for small values of β, but departs more from the truth as β increases, and possibly should be limited to values of β below some limit, say 45°, assumed arbitrarily.

* See Merriman's "Mechanics of Materials," p. 263: also "Stresses in Masonry Dams," by the writer, *Transactions*, Am. Soc. C. E., Vol. LXIV, pp. 220-221. In this article, the directions and amounts of the principal normal stresses, at various points of a horizontal section of a dam, are computed.

† *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, Session, 1907-1908, Part II.

‡ The same state of affairs exists in the so-called "beams of equal strength," having variable depth, where the ordinary theory of the books is plainly inadequate.

When the external force of Fig. 1 acts downward, the compressive area is below the neutral axis, and the reinforcement will be placed parallel to and near M N. All the compressive stresses now act parallel to the lower face, as in the common theory.

The usual hypotheses, that no tension exists in the concrete on the reinforced side of the neutral axis, and that plane sections before stress will remain plane sections after stress, will be adopted; but the latter hypothesis cannot be expected to apply very closely when β is large, particularly when it is near 90 degrees.

A general solution will now be given which includes every possible case. All special cases can be at once derived from this general solution.

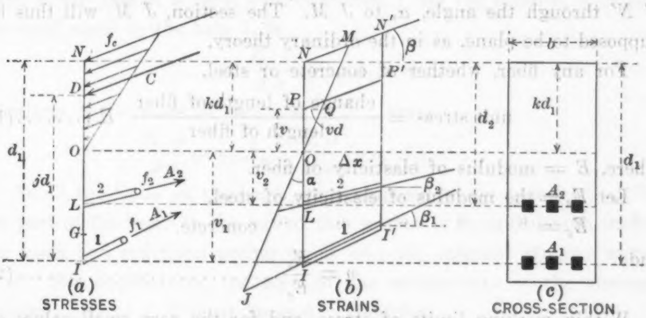

FIG. 2.

In (*a*) and (*b*), Fig. 2, are shown two longitudinal sections, and in (*c*), a section of a part of a beam subjected to stress. I N represents a section of the beam taken always parallel to the direction of the loads, which may be weights, soil reactions, earth thrusts, etc. The shear due to the loads thus acts along I N, and the moment is the same for any point of the section.

For the breadth, b, let A_1, A_2, . . ., represent the areas of the cross-sections (taken at right angles to the axes) of the bars, 1, 2, . . ., at depths d_1, d_2, . . ., respectively.

Let f_1, f_2, . . ., represent the unit stresses in the bars, 1, 2, . . ., so that the total stresses in the successive layers of bars, 1, 2, . . ., for the breadth, b, are f_1 A_1, f_2 A_2, . . ., respectively.

The angles, β, β_1, β_2, . . . (expressed in radians), are those made by the surface, N N', bars 1, bars 2, . . ., respectively, with the normal to the section, I N.

Let O be the neutral axis and D the point where the resultant, C, of the compressive forces (all acting parallel to $N\,N'$) meets $I\,N$.

Let $N\,O = k\,d_1$, $D\,I = j\,d_1$, $O\,I = v_1$, $O\,L = v_2$, etc.

$I'\,N'$ represents a section parallel to $I\,N$, Fig. 2 (*b*), and at a perpendicular distance, $\Delta\,x$, from it. The "fiber," $P\,P'$, parallel to $N\,N'$, of concrete, will be supposed to have an area equal to the section made by the plane $I\,N$, $\Delta\,a$; and the distance, $O\,P$, will be called v. Thus the area of a right section of the fiber is $\Delta\,a \cos.\,\beta$; and, if f is the unit stress on a right section of the fiber, $P\,P'$, the total compressive stress on the fiber will be $(f\,\Delta\,a \cos.\,\beta)$.

After strain, suppose the plane section, $I\,N$, rotates relatively to $I'\,N'$ through the angle, α, to $J\,M$. The section, $J\,M$, will thus be supposed to be plane, as in the ordinary theory.

For any fiber, whether of concrete or steel,

$$\text{unit stress} = \frac{\text{change of length of fiber}}{\text{length of fiber}}\;E \ldots\ldots\ldots (1)$$

where, E = modulus of elasticity of fiber.

Let E_s = the modulus of elasticity of steel,

E_c = " " " " " concrete,

and,

$$n = \frac{E_s}{E_c} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Within working limits of stress, and for the very small values of α corresponding to very small values of $\Delta\,x$, the change of length of $P\,P' = P\,Q = v\,\alpha \sec.\,\beta$, very nearly*; hence the unit stress, f, on this fiber, $P\,P'$, by Equation 1 is

$$f = \frac{P\,Q}{P\,P'}\,E_c = \frac{v\,\alpha \sec.\,\beta}{\Delta\,x \sec.\,\beta}\,E_c = \frac{v\,\alpha}{\Delta\,x}\,E_c.$$

Hence, as this unit stress on a right section of the fiber acts on the right sectional area, $\Delta\,a \cos.\,\beta$, the total stress on the fiber is

$$\frac{v\,\alpha}{\Delta\,x}\,E_c\,(\Delta\,a \cos.\,\beta).$$

and its component perpendicular to $I\,N$ is

$$\frac{E_c\,\alpha \cos.^2 \beta}{\Delta\,x}\,(v\,\Delta\,a).$$

* In the triangle, $P\,O\,Q$, by the law of sines, $P\,Q = \dfrac{v \sin.\,\alpha}{\cos.\,(\alpha + \beta)}$. As α tends toward zero, this approaches $\dfrac{v\,\alpha}{\cos.\,\beta} = v\,\alpha \sec.\,\beta$.

The sum of such components on the compressed area of depth, $k\, d_1$, and breadth, b, is

$$\frac{E_c\, \alpha \cos.^2 \beta}{\Delta x} \sum_0^{kd_1} (v\, \Delta\, a) = \frac{E_c\, \alpha \cos.^2 \beta}{\Delta x} \left(\frac{1}{2}\, b\, k^2\, d_1^2 \right),$$

since $\sum_0^{kd_1} (v\, \Delta\, a) =$ the statical moment of the area under compression about the neutral axis $= b\, k\, d_1 \frac{1}{2}\, k\, d_1$.

For the layer of bars 1, at I, similarly, the unit stress is

$$f_1 = \frac{J\, I}{I\, I'} E_s = \frac{v_1\, \alpha \sec.\, \beta_1}{\Delta x \sec.\, \beta_1} E_s = \frac{E_c\, \alpha}{\Delta x} (n\, v_1).$$

Also, as the area of a right section of bars 1, for the breadth, b, is A_1, the total stress in the bars 1, is

$$\frac{E_c\, \alpha}{\Delta x} (n\, v_1\, A_1).$$

Similarly, the stresses in bars 2, 3, . . ., are

$$\frac{E_c\, \alpha}{\Delta x} (n\, v_2\, A_2),\ \frac{E_c\, \alpha}{\Delta x} (n\, v_3\, A_3),\ . . .$$

As all the loads on the beam were supposed to act parallel to $I\, N$, the part of the beam to the left of this section is in equilibrium under the loads and reactions acting on it and the internal stresses along $I\, N$. For equilibrium, the sum of the components of the stresses perpendicular to $I\, N$ must be zero. Therefore,

$$\frac{E_c\, \alpha}{\Delta x} \cos.^2 \beta \left(\frac{1}{2}\, b\, k^2\, d_1^2 \right)$$
$$= \frac{E_c\, \alpha}{\Delta x}\, n\, (v_1\, A_1 \cos.\, \beta_1 + v_2\, A_2 \cos.\, \beta_2 + . . .).$$

From Fig. 2,

$$v_1 = d_1 - k\, d_1,\ v_2 = d_2 - k\, d_1,\ . . .;$$

hence, on substituting these values, striking out the common factor, and reducing, we derive,

$$\frac{1}{2} \cos.^2 \beta\, b\, d_1^2\, k^2 + n\, d_1\, (A_1 \cos.\, \beta_1 + A_2 \cos.\, \beta_2 + . . .)\, k$$
$$= n\, (d_1\, A_1 \cos.\, \beta_1 + d_2\, A_2 \cos.\, \beta_2 + . . .) \ldots\ldots\ldots (3)$$

From this quadratic in k, the value of k is computed, and thus $k\, d_1 = O\, N$ can be found and the neutral axis located. Also, as the compressive stresses are uniformly varying, $D\, N = \frac{1}{3}\, O\, N$; therefore, $j\, d_1 = d_1 - \frac{1}{3}\, k\, d_1$; and

$$j = 1 - \frac{1}{3}\, k \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

The Resisting Moment, M_s, of the Steel.—The moment, M, at the section, $I\ N$, due to the external forces, is equal to the resisting moment of the stresses acting along the section. Calling the perpendicular distances from D to bars 1, 2, . . ., p_1, p_2, . . ., respectively,

$$M_s = f_1\ A_1\ p_1 + f_2\ A_2\ p_2 + \ldots, \ldots\ldots\ldots\ldots (5)$$

where, $p_1 = j\ d_1 \cos. \beta_1$, $p_2 = D\ L \cos. \beta_2$, etc.

Now, as $f_1 = \frac{E_c\ \alpha}{\Delta\ x}\ n\ v_1$, $f_2 = \frac{E_c\ \alpha}{\Delta\ x}\ n\ v_2$, . . ., it follows that

$$f_2 = \frac{v_2}{v_1}\ f_1,\ f_3 = \frac{v_3}{v_1}\ f_1, \ldots, \ldots\ldots\ldots\ldots (6)$$

or the unit stresses in the bars vary directly with the distances from the neutral axis. The unit stresses in the interior bars will thus always be less than f_1, so that such an arrangement of bars is uneconomical.

On substituting Equation 6 in Equation 5,

$$M_s = f_1\ \left(A_1\,p_1 + \frac{v_2}{v_1}\ A_2\,p_2 + \frac{v_3}{v_1}\ A_3\,p_3 + \ldots\right) \ldots\ldots\ldots (7)$$

If preferred, after locating the point, D, on a drawing, the perpendiculars, p_1, p_2, . . ., can be measured to scale; otherwise, they may be computed readily by the formulas given.

If the resisting moment of the steel, M_s, is less than that of the concrete, M_c, for assigned maximum unit stresses, then the moment, M, of the external forces is put equal to the right member of Equation 7, the value of f_1 ascertained, and, from Equation 6, the values of f_2, f_3, . . ., are computed. The stresses in bars 1, 2, . . ., are thus $f_1\ A_1$, $f_2\ A_2$, . . .

Otherwise, if a certain value is assigned to f_1, as 16 000 lb. per sq. in., and A_2, A_3, . . ., are assumed, from Equation 7, A_1 can be computed. For rough computations, A_2, A_3, . . ., may often be ignored, in which case we can write,

$$M_s = f_1\ A_1\ p_1 = f_1\ A_1\ j\ d_1 \cos. \beta_1.$$

The Resisting Moment, M_c, of the Concrete.—To compute M_c, the position of the resultant, R, of the stresses in the bars, must first be found. The magnitudes of the forces acting on the bars 1, 2, 3, . . ., are

$$f_1\,A_1,\ f_1\,\frac{v_2}{v_1}\ A_2,\ f_1\ \frac{v_3}{v_1}\ A_3, \ldots,$$

and, as these are proportional to f_1, the direction and line of action of R are the same for any value of f_1 and hence for $f_1 = 1$.

Let $H =$ the component of R perpendicular to $I\,N$ when $f_1 = 1$; then

$$H = A_1 \cos.\,\beta_1 + \frac{v_2}{v_1} A_2 \cos.\,\beta_2 + \cdots$$

Suppose the resultant cuts $I\,N$ at G; then, taking moments about D,

$$H \cdot D\,G = A_1 p_1 + \frac{v_2}{v_1} A_2 p_2 + \frac{v_3}{v_1} A_3 p_3 + \cdots$$

The right member, presumably, has already been computed in applying Equation 7; hence $D\,G$ is quickly ascertained and the point, G, located.

Call the maximum unit stress on the concrete at N, f_c; the unit stress on the fiber, $P\,P'$, at P is thus $\frac{f_c}{k\,d_1}\,v$. This acts on the area $(\Delta\, a \cos.\,\beta)$; hence the stress on the fiber is

$$\frac{f_c}{k\,d_1}\, v\, \Delta\, a \cos.\,\beta,$$

and the sum of such stresses is

$$C = \frac{f_c}{k\,d_1} \cos.\,\beta \sum_0^{kd_1} (v\,\Delta\, a) = \frac{1}{2}\,\frac{f_c}{k\,d_1} \cos.\,\beta\, b\,(k\,d_1)^2.$$

Therefore, $C = \frac{1}{2} f_c\, b\, k\, d_1 \cos.\,\beta$.

Taking moments about G,

$$M_c = \frac{1}{2} f_c\, b\, k\, d_1 \cos.^2 \beta\, \overline{D\,G} \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

If, for assigned maximum values of f_1 and f_c, $M_s > M_c$, the beam is over-reinforced and M is placed equal to the right member of Equation 8, as its strength is now limited by that of the concrete. This case rarely occurs in practice.

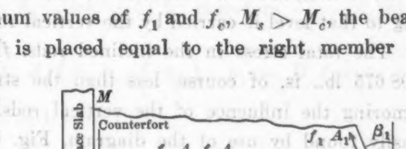

FIG. 3.

As an illustration of the application of the formulas, take the counterfort, $B\,M$, Fig. 3, attached to the face-slab, $N\,M$, and suppose the counterfort to be subjected to a horizontal earth thrust of 123 750 lb., acting to the left and 9.06 ft. above N, giving a bending moment at N of 13 454 100 in-lb. The section, $N\,B$, corre-

sponding to $N\ I$ of Fig. 2, is taken parallel to the load (the earth thrust), and is therefore horizontal.

The width of the counterfort is $b = 18$ in. The inclined bars have a total sectional area $= A_1 = 9.45$ sq. in., and the vertical bars, a common area, $A = 0.784$ sq. in. Using the foregoing notation, $\beta_1 = 23° 58'$, $\beta_2 = \beta_3 = \ldots = \beta_7 = 0$. The distances, d, were measured from N, or from the front face of the vertical slab: $d_1 = 128$, $d_2 = 108$, $d_3 = 100$, $d_4 = 92$, $d_5 = 76$, $d_6 = 60$, $d_7 = 44$, all in inches. Assume $n = 15$.

On substituting known values in Equation 3, we derive $k = 0.311$. Therefore, $j = 1 - \frac{1}{3}\ k = 0.896$; whence $N\ O = k\ d_1 = 39.8$ in., $D\ B = j\ d_1 = 114.7$ in.; also, $N\ D = \frac{1}{3}\ N\ O = 13$ in.

On subtracting $N\ O = 40$ from $d_1, d_2, \ldots$, we derive v_1, v_2, etc. The perpendiculars, $p_1, p_2, \ldots$, from D on bars 1, 2, . . ., are $p_1 = j\ d_1 \cos. \beta_1 = 105$, $p_2 = d_2 - N\ D = 95$, . . ., respectively.

From Equation 7 we have, $13\,454\,100 = 1\,164\ f_1$; whence $f_1 = 11\,500$ lb. per sq. in.; from Equation 6, $f_2 = 8\,970$, . . ., $f_7 = 575$ lb. per sq. in.

As the weight of the heel-slab must be carried by the rods, the areas and spacing of the vertical rods were designed to carry their proportionate part of the weight of the heel-slab. The stresses corresponding are found to be in excess of those due to the moment, M. This excess is taken up by the bond stress in a short distance above $N\ B$, so that, above a certain level, only the moment stress corresponding to that level is carried by the vertical rods.

The total stress in the inclined rods, $f_1\ A_1 = 11\,500 \times 9.45 = 108\,675$ lb., is, of course, less than the stress, 127 000 lb., found by ignoring the influence of the vertical rods. This last stress is most easily found by use of the diagram, Fig. 6, and Equation 11, given later.

This example of the counterfort has been given more for the purpose of gaining an idea of the actual stresses involved than of urging the adoption of the more refined method in practice.

If the diameter of a bar is to be some multiple of $\frac{1}{8}$ in., there may be no saving by the use of the more exact method. It must be remembered, too, that the vertical bars are not always bonded securely in the base-slab, the earth thrust may also be much increased in times

of heavy rains, where the filling is not adequately drained; and, further, the foundation may be more yielding than estimated.

In the next example, Fig. 4, representing the heel of a T-wall with a fillet, the wall being on the point of overturning, the exact method seems advisable.

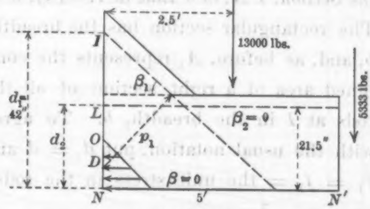

FIG. 4.

The total moment at the section, $N\ I$, due to the two forces shown, is

$$M = (13\,000 \times 2.5 + 5\,333 \times 5)\ 12 = 709\,992 \text{ in-lb.}$$

The reinforcement, shown by the broken lines, for both inclined and horizontal bars, consists of $\frac{7}{8}$-in. square bars, 8 in. from center to center, corresponding to $A_1 = A_2 = 1.15$ sq. in. for a breadth of $b = 12$ in.

Assuming $n = 15$, $\beta_1 = 43°\ 10'$, $\beta = \beta_2 = 0$, $d_1 = 42$, $d_2 = 21.5$, and substituting in Equation 3,

$$\frac{1}{2}\ b\ d_1{}^2\ k^2 + n\ A_1\ d_2\ (\cos.\ \beta_1 + 1)_k = n\ A_1(d_1 \cos.\ \beta_1 + d_2)$$

we find, after solution,

$$k = 0.238\ ;\ \text{therefore } j = 1 - \frac{1}{3}\ k = 0.921,$$

$$N\ O = k\ d_1 = 10 \text{ in., } D\ I = j\ d_1 = 39 \text{ in., } N\ D = 3.33 \text{ in.,}$$

$$v_1 = d_1 - kd_1 = 32,\ v_2 = d_2 - kd_1 = 11.5\ ;\ \text{therefore } \frac{v_2}{v_1} = 0.361.$$

By Equation 7, $M_s = f_1\ A_1\left(p_1 + \frac{v_2}{v_1}\ p_2\right)$, $p_1 = D\ I \cos.\ \beta_1$

$$= 28.4,\ p_2 = 18.2;$$

$$\text{therefore } M = 710\,000 = 1.15\ (18.2 + 6.57)\ f_1 = 40\ f_1;$$

therefore $f_1 = 17\,750$ lb. per sq. in., $f_2 = \frac{v_2}{v_1}\ f_1 = 6\,380$ lb. per sq. in.

Where the foundation is good, there can be only a very small moving over of the wall, so that the friction force, 5 333 lb., at N' can be neglected and the soil reaction included. In this particular example (not given in full here) the new values of f_1 and f_2, thus found, are only four-tenths of those given previously.

Special Case.—Where $\beta = 0$ and All the Rods Lie in One Plane Which Meets the Section, I N, in a Line at I.—Fig. 5.— The rectangular section has the breadth, b, and, as before, A_1 represents the combined area of a right section of all the rods at I in the breadth, b. To agree with the usual notation, put $d_1 = d$ and $f_1 = f_s =$ the unit stress in the rods. Equation 3 now reduces to

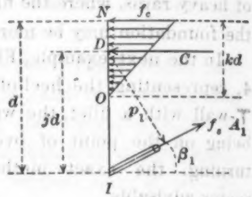

FIG. 5.

$$\frac{1}{2}\, b\, d^2\, k^2 + nd\, A_1 \cos.\, \beta_1\, k - nd\, A_1 \cos.\, \beta_1 = 0.$$

Placing $p = \dfrac{\text{steel area}}{\text{concrete area}} = \dfrac{A_1}{b\,d}$, and dividing by $\dfrac{1}{2}bd^2$,

$$k^2 + 2\, n\, p \cos.\, \beta_1\, k - 2\, n\, p \cos.\, \beta_1 = 0,$$

therefore $k = -\, n\, p \cos.\, \beta_1 + \sqrt{(n\, p \cos.\, \beta_1)^2 + 2\,(n\, p \cos.\, \beta_1)}$ ..(9)

when $\beta_1 = 0$, this reduces to the usual formula for prismatic beams,

$$k = -\, n\, p + \sqrt{(n\, p)^2 + 2\,(n\, p)} \quad\ldots\ldots\ldots\ldots(10)$$

In Fig. 6, the values of k and $j = 1 - \frac{1}{3}k$, are given as ordinates to the dotted curves for various percentages of steel and values of β_1, varying from 0° to 40°, assuming $n = 15$.

The resisting moment of the steel is found by taking moments about D,

$$M_s = f_s\, A_1 \cos.\, \beta_1\, j\, d = f_s\, A_1\, p_1 \quad\ldots\ldots\ldots\ldots(11)$$

where $p_1 =$ the perpendicular distance from D on the bar.

The resisting moment of the concrete is found by taking moments about I,

$$M_c = \frac{1}{2} f_c\, b\, k\, d\, j\, d = \frac{1}{2} f_c\,(k j)\, b\, d^2 \quad\ldots\ldots\ldots(12)$$

For assigned maximum working values of f_s and f_c, the least resisting moment is equated to the moment of the external forces.

In using Fig. 6, note that p is not the percentage of steel, but $\frac{1}{100}$ of the percentage.

Resuming the example of the counterfort, Fig. 3, what will be the result of ignoring the vertical rods? With $A_1 = 9.45$ sq. in. and as

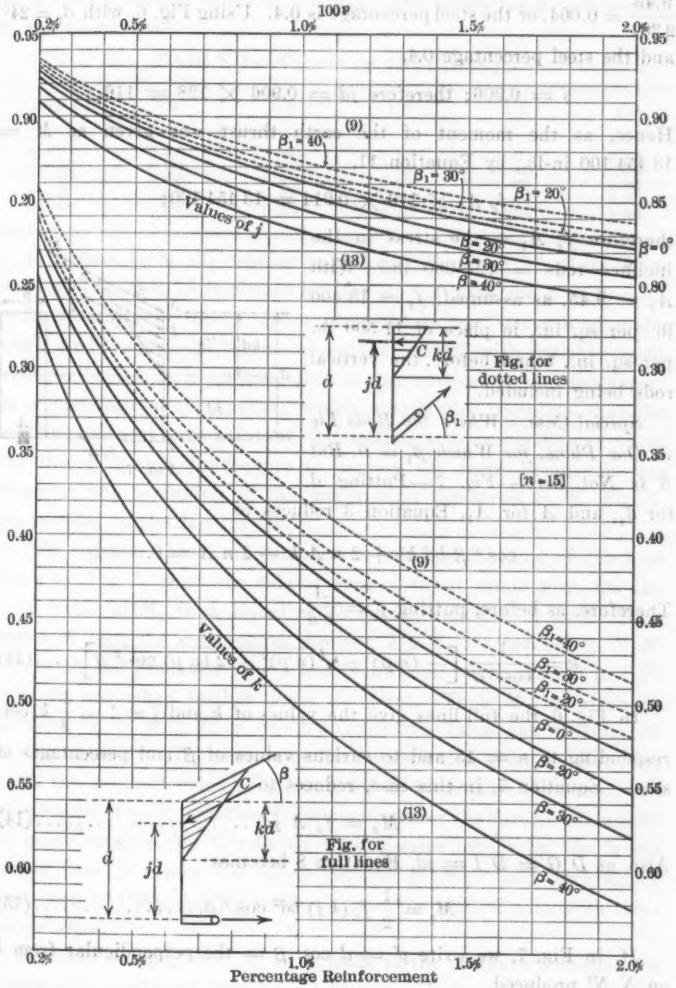
100p
0.2%
0.5%
1.0%
1.5%
2.0%
0.95
0.90
0.85
0.80
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0.55
0.60
(9)
$\beta_1 = 40°$
$\beta_1 = 30°$
$\beta_1 = 20°$
Values of j
$\beta = 0°$
$\beta = 20°$
$\beta = 30°$
$\beta = 40°$
(13)
C
kd
d
jd
β_1
Fig. for dotted lines
[n = 15]
(9)
$\beta_1 = 40°$
$\beta_1 = 30°$
$\beta_1 = 20°$
$\beta = 0°$
$\beta = 20°$
$\beta = 30°$
$\beta = 40°$
Values of k
β
C
(13)
kd
d
jd
Fig. for full lines
Percentage Reinforcement

FIG. 6.

the area of the base or section, $N\ B = 128 \times 18 = 2\,304$ sq. in., $p = \frac{9.45}{2\,304} = 0.004$, or the steel percentage is 0.4. Using Fig. 6, with $\beta_1 = 24°$ and the steel percentage 0.4,

$$j = 0.906;\ \text{therefore}\ jd = 0.906 \times 128 = 116.$$

Hence, as the moment of the earth thrust was given as $M = 13\,454\,100$ in-lb., by Equation 11,

$$f_s\ A_1 \times 116 \times 0.914 = 13\,454\,100;$$

therefore $f_s\ A_1$ = the stress in the inclined rods = 127 000 lb. With $A_1 = 9.45$, as assumed, $f_s = 13\,400$ lb. per sq. in., in place of 11 500 lb. per sq. in., found before, the vertical rods being included.

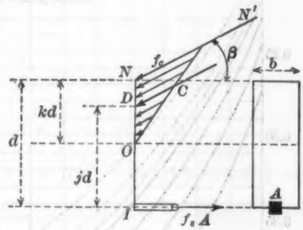

FIG. 7.

Special Case.—Where the Rods Lie in One Plane, for Which $\beta_1 = 0$, But β is Not Zero.—Fig. 7.—Putting d for d_1, and A for A_1, Equation 3 reduces to

$$\cos.^2 \beta\ bd\ k^2 + 2\ n\ A\ k - 2\ n\ A = 0.$$

Therefore, as before, putting $p = \frac{A}{b\ d}$,

$$k = \frac{1}{\cos.^2 \beta}\left[-(n\ p) + \sqrt{(n\ p)^2 + 2\ (n\ p)\cos.^2 \beta}\right] \ldots\ldots (13)$$

In Fig. 6, the full lines give the values of k and $j = 1 - \frac{1}{3}\ k$, corresponding to $n = 15$ and to various values of β and percentages of steel. Equation 7, in this case, reduces to

$$M_s = f_s\ A\ jd \ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

Also, as $D\ G = D\ I = jd$, Equation 8 becomes

$$M_c = \frac{1}{2}\ f_c\ (k\ j)\ bd^2 \cos.^2 \beta \ldots\ldots\ldots\ldots\ldots (15)$$

If, in Fig. 7, we write $p' = d \cos.\ \beta$ = the perpendicular from I on $N\ N'$ produced,

$$M_c = \frac{1}{2}\ f_c\ (k\ j)\ b\ p'^2 \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

The product, kj, varies with β. In Table 1 the values of $(kj) \cos.^2 \beta$, are given for various values of p and β. The values for $\beta = 0$, pertain to a prismatic beam.

TABLE 1.—VALUES OF $(k\,j) \cos.^2 \beta$.

$100\,p$	0.2	0.6	1.0	1.6	2.0
$\beta = 0$	0.200	0.303	0.359	0.411	0.441
$\beta = 10°$	0.199	0.299	0.352	0.403	0.427
$\beta = 20°$	0.186	0.281	0.329	0.376	0.397
$\beta = 30°$	0.169	0.251	0.294	0.333	0.351
$\beta = 40°$	0.145	0.212	0.246	0.276	0.289

It is seen from the tabular values and Equation 15, that M_c as given by Equation 15 is always less for $\beta > 0$ than for $\beta = 0$.

In the ordinary theory, given in textbooks on "strength of materials", for homogeneous "beams of equal strength" with variable depth, vertically loaded, it is assumed that at any vertical section the theory for a prismatic beam applies; which entails the postulate that the bending stress at any point of the section acts perpendicular to it. The theory is thus inadequate to express the facts, because it was shown, in the beginning of this paper, that the stress at N, Fig. 7, acts parallel to the face, $N\ N'$.

This common theory, if extended to reinforced beams of the type shown by Fig. 1, is on the side of danger, for it would give, for any β, the value of M_c from Equation 15 corresponding to $\beta = 0$. As a matter of fact, the compressive stresses, Fig. 7, act parallel to $N\ N'$ only at N, and gradually take a less inclination to the normal to $I\ N$, as points are taken farther down the joint. Thus M_c, as given by Equation 15 for $\beta > 0$, is less than the true value, and is thus on the side of safety. The true value lies between that given by Equation 15 for the assumed value of β, supposed to be greater than zero, and the value for $\beta = 0$, doubtless lying much nearer the former value than the latter because M_c is affected to a greater extent by the larger stresses near N, which are nearly parallel to $N\ N'$ and have longer arms, than by those smaller stresses nearer the neutral axis, with their shorter arms.

The case where $\beta = 0$, $\beta_1 = 0$, leads to the ordinary formulas for a prismatic reinforced beam, for which a valuable working diagram was first given* by Arthur W. French, M. Am. Soc. C. E.

* *Transactions*, Am. Soc. C. E., Vol. LVI, 1906, p. 362.

Maximum Shearing Stresses.—Let $I\ N\ N'\ I'$, Fig. 8, represent a part of the beam lying between two sections, $I\ N$, $I'\ N'$, perpendicular to the plane of the paper and parallel to the direction of the loads, and dx apart. These sections are rectangular, having the breadth b. The beam is supposed to be reinforced with the rods $I\ I'$, $L\ L'$, $S\ S'$, . . .

Let V and V' denote the shears due to the loading C and C', the resultants of the compressive stresses, and T and T', the resultants of the tensions in the bars, at the sections, $I\ N$, $I'\ N'$, respectively.

The part of the beam, $I\ N\ N'\ I'$, is in equilibrium under the action of these forces, with the directions given by the arrows, its own weight and load; but as the weight and load are equal to an expression that contains dx as a factor, it is infinitesimal, compared with the forces, and is neglected. It is to be observed, also, that T' is supposed to act in the same line as T, because it approaches this position indefinitely as dx tends toward zero.

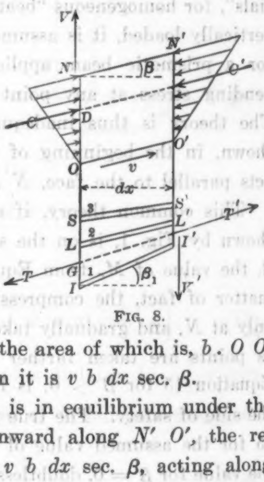

FIG. 8.

Suppose that T meets the section, $I\ N$, at G, and takes moments about G of the forces in equilibrium:

$$(C' - C) \cos. \beta\ \overline{D\ G} = V'\ dx.$$

Taking the neutral surface as parallel to the surface, $N\ N'$, and calling v the unit shear along this neutral surface, the area of which is, $b \cdot O\ O' = b\ dx \sec. \beta$, the total shearing stress on it is $v\ b\ dx \sec. \beta$.

The prism, $N\ O\ O'\ N'$, of breadth, b, is in equilibrium under the shears, acting upward along $O\ N$, downward along $N'\ O'$, the resultants, C, C', and the shearing stress, $v\ b\ dx \sec. \beta$, acting along $O\ O'$. Hence the algebraic sum of the horizontal components is zero. Therefore,

$$(C' - C) \cos. \beta = (v\ b\ dx \sec. \beta) \cos. \beta = v\ b\ dx.$$

Substituting this value in the preceding equation, dividing by dx, and then taking the limit, as dx approaches zero and V' approaches V indefinitely, it is found that

$$v = \frac{V}{b\ \overline{D\ G}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

The point, G, can be found without knowing the stresses in the bars, as explained previously. This unit shear, v, exists from the neutral surface to the bars nearest to it.

The case most frequently met is where all the steel reinforcement is placed in the plane through $I\ I'$, perpendicular to the plane of the paper. For this case, G coincides with I and $D\ G = j\ .\ I\ N = jd$, putting $I\ N = d$. Therefore,

$$v = \frac{V}{jd\ .\ b} \qquad (18)$$

From Fig. 6, j can be found when either β or β_1 is zero, or when both are zero. In the latter case, Equation 18 is a well-known formula, and for approximate values, $j = \frac{7}{8} = 0.875$ is often used.

The maximum shearing stress given by the last equation remains the same for points between the neutral surface and the steel. Above the neutral surface, it decreases, by the usual parabolic law, to zero at N. The unit shear, v, acts parallel to $N\ N'$.

Bond Stress.—In Fig. 8, first suppose the steel bars to lie only in the plane, $I\ I'$. For the breadth, b, let the total tension in the bars at I be t_1, acting in the direction $I'\ I$, and the corresponding tension at I' be t_1', acting in the direction $I\ I'$. The increment of stress $(t_1' - t_1)$, is transmitted by the bond between the concrete and the steel.

Let $u =$ this bond stress per square inch of surface of the rods;

$o =$ the surface area of one bar for 1 in. of length (equal to the perimeter);

$\Sigma\, o =$ the surface area of all the bars in the width, b, for 1 in. of length.

In the length, $I\ I' = dx$ sec. β_1, the total area is $I\ I'\ \Sigma\ o$, and the total bond stress is $u\ dx$ sec. $\beta_1\ \Sigma\ o = t_1' - t$.

Taking moments about D, of the forces, V, V', C, C', t_1, t_1', in equilibrium,

$$(t_1' - t) \cos.\ \beta_1\ I\ D = u\ dx\ \Sigma\ o\ I\ D = V'\ dx.$$

Dividing by dx, taking limits as dx approaches zero, and replacing $I\ D$ by $j\ d$,

$$u = \frac{V}{j\ d\ \Sigma\ o} \qquad (19)$$

In this formula, $d = N I$, and j is to be found by the diagram, Fig. 6, or by previous methods. When $\beta_1 = 0$, $\beta = 0$; the approximate value, $\frac{7}{8}$, is often used.

The formulas for both shear and bond stresses, Equations 18 and 19, are of the usual form, corresponding to $\beta_1 = \beta = 0$. They are more general than for the latter case, and it is only necessary to substitute the proper value of j for the values of β and β_1 assumed to obtain the proper values of the stresses.

Where there are several layers of bars, as at $I\,I'$, $L\,L'$, $S\,S'$, . . ., let u_1, u_2, . . ., indicate the unit bond stresses and $\Sigma\, o_1$, $\Sigma\, o_2$, . . ., the areas per linear inch of surface of the rods, for the width, b, for the respective bars; then the total bond stresses at $I\,I'$, $L\,L'$, . . ., are

$$u_1\, I\,I'\, \Sigma\, o_1,\ u_2\, L\,L'\, \Sigma\, o_2,\ \ldots,$$

which are equal, respectively, to $(t_1' - t_1)$, $(t_2' - t_2)$, . . ., the subscripts referring to bars 1, 2, . . ., at $I\,I'$, $L\,L'$, ...

Taking moments about D and proceeding as before, we easily derive,

$$\overline{I\,D}\, u_1\, \Sigma\, o_1 + \overline{L\,D}\, u_2\, \Sigma\, o_2 + \overline{S\,D}\, u_3\, \Sigma\, o_3 + \ldots = V.$$

Now, the unit bond stress in any rod is proportional to the unit elongation of the rod, or to its unit stress, which varies with the distance from O (See Equation 6); hence, for the same loading,

$$u_2 = u_1 \frac{O\,L}{O\,I},\ u_3 = u_1 \frac{O\,S}{O\,I},\ \ldots \qquad (20)$$

Therefore the previous equation reduces to

$$u_1 \left[I\,D\, \Sigma\, o_1 + L\,D\, \frac{O\,L}{O\,I}\, \Sigma\, o_2 + S\,D\, \frac{O\,S}{O\,I}\, \Sigma\, o_3 + \ldots \right] = V \quad (21)$$

From this equation, u_1 is found; then, from Equation 20, u_2, u_3, . . .

If the interior bars are ignored, and u_1 is found from the simple formula, Equation 19, to be within safe limits, it follows that the true bond stresses on all the bars are less, and are therefore within safe limits.

The application of Equations 18 and 19 is obvious. As Equations 17 and 21 are unusual forms, it may prove of service to the computer to give a numerical illustration. Let it be proposed, therefore, to find

the shear and bond stresses for the heel-slab with the fillet, Fig. 4, already considered.

The total shear at $N\,I$, for $b = 12$ in. length of wall, is $V = 18\,333$ lb. The tension in the bar which makes the angle, $\beta_1 = 43^\circ\ 10'$, with the horizontal, is $f_1\ A_1$, and that in the horizontal bar is $f_1 \dfrac{O\,L}{O\,I} A_2 = f_1 \dfrac{11.5}{32} A_2 = 0.361\ f_1\ A_2$. As $A_1 = A_2$, these tensions are in the ratio, $1:0.361$, and the point, G, where the resultant of the two tensions cuts $N\,I$, is the same as for two forces of magnitudes 1 and 0.361, having the same positions and directions.

Let $H =$ the sum of the horizontal components of the two forces supposed. Therefore

$$H = 1 \times \cos.\ \beta_1 + 0.361 = 1.090.$$

Taking moments about D,

$$\begin{aligned} H\,.\ \overline{D\ G} &= 1 \times \cos.\ \beta_1 \times \overline{D\,I} + 0.361 \times \overline{D\,L} \\ &= 0.729 \times 39 + 0.361 \times 18.2 = 34.97. \end{aligned}$$

Therefore, $D\ G = 32.1$ in., and the maximum shear is

$$v = \frac{V}{b\,.\,D\ G} = \frac{18\,333}{12 \times 32.1} = 47.6 \text{ lb. per sq. in.}$$

Both the inclined and horizontal reinforcement consists of ⅞-in. square rods, spaced 8 in. from center to center. Therefore,

$$\Sigma\ o_1 = \Sigma\ o_2 = \frac{12}{8}\ (3.5) = 5.25 \text{ for } b = 12 \text{ in.}$$

Also, from previous computations,

$$I\ D = 39,\ L\ D = 18.2,\ O\ L = 11.5,\ O\ I = 32 \text{ in.}$$

Therefore, by Equation 21, the unit bond stress, u_1, on the inclined rods is given by

$$u_1 \times 5.25 \left[39 + \frac{11.5 \times 18.2}{32} \right] = 18\,333.$$

Therefore, $u_1 = 76.7$ lb. per sq. in., and the unit bond stress on the horizontal rods is

$$u_2 = \frac{O\ L}{O\ I}\ u_1 = \frac{11.5}{32} \times 76.7 = 27.5 \text{ lb. per sq. in.}$$

In what precedes, there is much that may appear novel. The problems that occur in practice pertain to two different classes of beams.

In those of the first class, illustrated by Figs. 3 and 4, the section, *I N*, is taken perpendicular to the face on the compressive side, and the bending stresses there act perpendicularly to the section, as in the ordinary theory. In beams of the second class, as in Fig. 1, the section, *I N*, is not perpendicular to the face on the compressive side, and the compressive stresses are all assumed to act parallel to the face. It is possible that this assumption, alone, will be open to criticism, for, accepting the hypothesis, all the results follow readily from simple mechanical laws.

The writer invites a careful criticism of this hypothesis, with the accompanying deductions. A strict solution of beams of the second class is doubtless impracticable, but the writer believes that he has effected a practical solution which is on the side of safety and may commend itself to the practitioner.

DISCUSSION

A. C. JANNI, M. AM. SOC. C. E. (by letter).—Before entering into any discussion on the opportunity and legitimacy of the assumption on which this work hinges, the writer wishes to call the author's attention to a little mathematical "license" which he took in simplifying the expression, $P\,Q = \frac{v \sin.\alpha}{\cos.(\alpha + \beta)}$,* because it undermines, perhaps, the whole mathematical "make-up" of all the formulas which depend on that *"lapsus calami"*. Mr. Janni.

There is no doubt that, as α tends toward zero, $\sin.\alpha$ may be substituted for α; this simplification being entirely legitimate; but, if α tends toward zero, the expression $\cos.(\alpha + \beta)$ does not tend toward $\cos.\beta$, by any means.

The author knows that $\cos.(\alpha + \beta)$ is purely a symbol of a certain operation to be performed, and is not an actual value; its value is given by $\cos.\alpha \cos.\beta - \sin.\alpha \sin.\beta$, and it is on this latter that he may operate his simplification. Thus his $P\,Q = v\,\alpha \sec.\beta$ should become $P\,Q = \frac{v\,\alpha}{\cos.\beta - \alpha \sin.\beta}$, when it is assumed that α tends toward zero, to be a mathematically correct simplification.

Concerning the assumption adopted by the author in order to find a "workable formula" for the compressive stress in concrete, and also some of his statements, the writer cannot quite agree. Two questions are before him, namely:

1. Do we need that formula?
2. Is that formula worked out correctly?

It is the writer's opinion that neither of these questions can be answered in the affirmative.

Theory is quite competent to take care of a design of a wedge-shaped beam, if necessary, despite what the author says. If the cross-section of a wedge-shaped concrete beam is rectangular, and the angle, β, is not large, the common formula for bending is sufficiently good to give the maximum compressive stress in the concrete.

It is a fact that the formula entails the truth—which, by the way, is not a postulate—demonstrated by Saint-Venant, that all bending stresses at any point act perpendicularly to the cross-section; but this, in the case where β is not very large, does not prevent the formula from giving a result sufficiently near to the truth to insure the maximum stress.

That the direction of the maximum stress, in the case of Fig. 1, is parallel to the top, $N\,M$, of the beam, and on which the author lays

* See foot-note, p. 748.

Mr. Janni.

so much stress, is, nowadays, mathematically ascertained.* In the case of a dam design, that direction is of capital importance, and the failure to recognize it, or the overlooking of it, in the design of some dams, has often been the cause of loss of thousands of innocent lives and, perhaps, millions of dollars. In a wedge-shaped beam, however (or in a counterfort), the conditions are such that the direction of the maximum compressive stress can be ignored in most cases.

If the cross-section of the wedge-shaped beam is not rectangular, then, with proper limitations, the usual formula to determine the principal stresses, reported in many textbooks, can be used. This formula is, precisely, designed for a beam by taking as unknown quantities certain inclined stresses, called "principal stresses", instead of the normal stresses in that section, and, really, this is the most logical way of designing a beam, although the common practice, for opportune reasons, follows that of the bending and shear formulas.

Winkler, in 1867, as far as the writer knows, wrote on this subject, and, after him, Culmann, Ritter, Guidi, and others treated the same question; so that the conception cannot appear novel, as the author seems to think, at least judging by some of his remarks.

If, finally, the inclination of *N M*, Fig. 1, is large, then Grashof's formula, with due limitations for the case of reinforced concrete, may be used.

In view of all the foregoing considerations, the writer thinks that there is no need of a formula for the case in point, and that, after all the arbitrary assumptions of the author in reaching his "workable formula", admitting its correct mathematical "make-up", its results are much farther from the truth than those given by the existing formulas, which, in all cases, are very reliable.

It is necessary, accordingly, to take into consideration the author's assumption in finding that formula, and, as a consequence, its mathematical reliability.

As the writer said before, it is very well known that the extreme top compressive stress is parallel to the beam top as, disregarding the friction on the soil, Fig. 1, the maximum tensile stress is parallel to the bottom, *L I*.

The author states that the maximum tensile stress at that region of the beam would be parallel to *L I*, if the beam were a homogeneous one.

Even if the beam were homogeneous, if friction is not disregarded, the maximum tensile stress, at the bottom of the beam, is not parallel to the bottom face.

When the beam is heterogenous, as in the author's case, equilibrium conditions do not change, if friction is included; the only dif-

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* M. Levy, *Annales des Ponts et Chaussées*, 1897; and F. Platzmann, *Ueber den Querschnitt der Staumauern*, Leipzig, 1908.

Mr. Janni.

ference being that, as usual, the tension in concrete is disregarded, so that the designer is inclined to disregard its direction, also, unless it is not a case of scientific research.

Now, the maximum compressive stress being parallel to the top, and the maximum tensile stress (always disregarding friction) being parallel to the bottom, the intermediate bending stresses, the author says, change their direction gradually from $N\,M$ to $L\,I$, Fig. 1. As a matter of fact, those intermediate bending stresses do not change their direction gradually in the way the author means. Fig. 9 gives an idea of the behavior of the internal stresses in the cross-section, $N\,I$, on account of the angle β. The author, however, states that, for the purpose of finding "workable formulas", he must assume that the intermediate compressive bending stresses be parallel to the maximum compressive bending stress. Such an assumption cannot be justified, when it is well known how those stresses act, and a designer knows how to determine them. Their direction, which is not constant for all, is a function of the angle, β, and not of cos. β, and their position is farther and farther from the author's assumption as β increases.

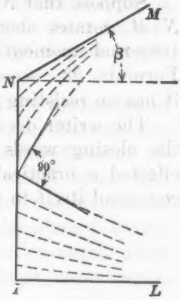

FIG. 9.

Apart from this assumption made by the author, which does not seem justifiable to the writer from any point of view, it is interesting to study the formula itself.

Formula 8, which, according to the author, gives the maximum compressive stress in the concrete, will be taken into consideration. For the sake of clearness, the writer will compare the theoretical formula for bending in a beam of constant cross-section, which, when β is not very large, may be used without any fear of going astray, with the author's formula for the same kind of stresses in a wedge-shaped beam, using for both the same lettering, Fig. 10.

The theoretical formula for bending in a beam of constant rectangular cross-section is

$$f_c = \frac{2\,M}{b\,y\left(h - \frac{1}{3}\,y\right)} = Q \quad \ldots\ldots\ldots\ldots\ldots (22)$$

The author's formula for bending in a wedge-shaped beam is

$$f_c = \frac{2\,M}{b\,y\left(h - \frac{1}{3}\,y\right)\cos.^2\beta} = \frac{Q}{\cos.^2\beta} \quad \ldots\ldots\ldots (23)$$

It is plain, as the author says, that Formula 23, for $\beta = 0°$, becomes Formula 22.

Mr. Janni. Formula 23, however, as given by the writer, presents some other information rather interestingly.

Suppose that $N\ I$, Fig. 10, be a constant section, and that the line, $N\ M$, rotates about N, the angle, β, increasing gradually while M (external moment), keeps constant, the section, $N\ I$, according to Formula 23, becomes weaker and weaker, until, when $\beta = 90°$, it has no resisting power whatever.

The writer does not think any comment necessary, except to recall the closing words of the paper: "The writer believes that he has effected a practical solution which is on the side of safety and may commend itself to the practitioner."

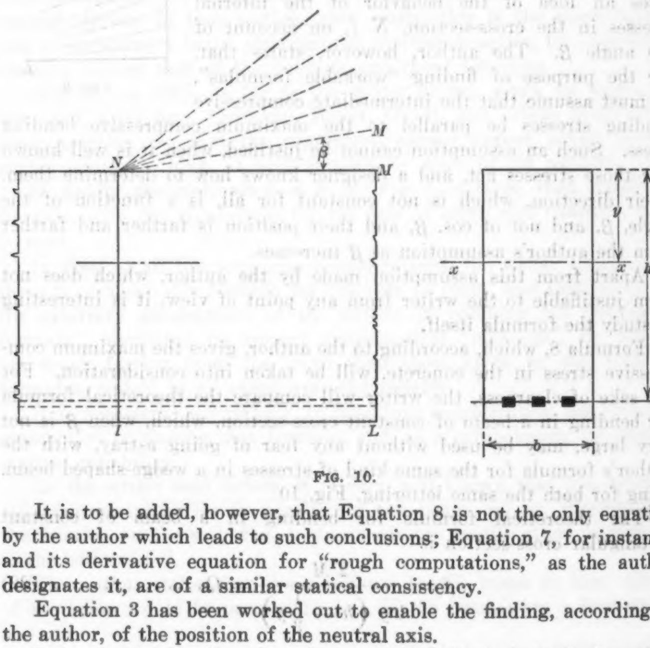

FIG. 10.

It is to be added, however, that Equation 8 is not the only equation by the author which leads to such conclusions; Equation 7, for instance, and its derivative equation for "rough computations," as the author designates it, are of a similar statical consistency.

Equation 3 has been worked out to enable the finding, according to the author, of the position of the neutral axis.

It is not clear to the writer, however, why the author gives a rather long equation (which, by the way, is not correct), when there are already several plain methods of finding that axis in a reinforced concrete beam.

The writer thinks it would be necessary for the author to devote a few words in justification of his point of view concerning Equation 3.

The writer, in closing, would like to ask the author the reason for his statement that "The unit stress, v, acts parallel to $N\ N'$". The writer has the impression that this statement, as put by the author, is not another of his assumptions, but a true and proper theorem concerning the equilibrium of internal forces in a wedge-shaped beam; and, as such, he would like further explanation. Mr. Janni.

Mr. Mensch.

L. J. MENSCH, M. AM. SOC. C. E. (by letter).—In the discussion of a paper on reinforced concrete construction before this Society several years ago, the statement was made that the counterforts of retaining walls do not act as beams. This alone would show that Mr. Cain's paper is important. It also seems that the writers of textbooks have neglected to devolop theories of wedge-shaped beams, although quite a number of such books contain theories nearly identical with that of Mr. Cain, for curved beams such as crane hooks, arched girders, etc.

Mr. Cain points out that an engineer is apt to overlook the fact that the compressive forces must be multiplied by $\cos.^2\beta$ when entered in the elastic equation, instead of by $\cos.\beta$ which one would be likely to do without going deeper into the matter. Probably a great many mistakes are made on this line, yet it is strange that they are made for the compression side of the beam, though on the tension side it is quite natural to multiply the actual cross-section (which is the $\cos.\beta$ value of the section made by the plane, $I\ N$) by $j\ d\ \cos.\beta$ to find the moment of the tensile forces.

For compression members, in trusses with inclined members, engineers are also accustomed to consider only the cross-section at right angles to the direction of the stress; therefore, it appears to the writer that only carelessness or incompetence is the cause of the mistake complained of. Mr. Cain's theory stands and falls with the fact that plane sections remain plane after deformation.

His assumption that the compressive forces are parallel to the compression face is certainly permissible in all cases where the percentage of reinforcement is low, say, less than 1%, and errs only slightly on the side of danger.

The law or assumption of plane sections remaining plane after deformation is certainly one of the greatest inventions of modern engineering, and enabled us to obtain a great number of simple working formulas, where formerly rules of thumb were used, yet the law is not universal, and is strictly proven only for straight beams of high-grade iron of rectangular section, within the elastic limit, and never for the neighborhood of ultimate load.

Professor Schule, of Zürich, about 12 years ago, proved that this assumption does not hold good for reinforced concrete beams, even under light loads, and why engineers all over the world have utterly disregarded these important tests is certainly strange.

Mr. Mensch.

It is a well-known fact that the plane-section theory for plain concrete beams gives results which vary just 100% from actual tests, and still the same theory is applied to reinforced concrete beams. R. L. Humphrey, M. Am. Soc. C. E., in the discussion of his painstaking tests of 343 reinforced concrete beams, declares that the compressive stresses in the concrete and the tensile stresses in the reinforcement often differed 50% from the theory. In the face of such facts it may be expected that Mr. Cain's table will differ just as much from facts, although it is much better than relying on a mere guess.

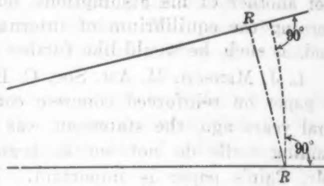

FIG. 11.

Tests of cast-iron beams prove that such wedge-shaped beams fail on a curved surface which is tangent to perpendicular lines to both faces of the beams, as shown in Fig. 11. For this reason the writer has advised* calculating such a counterfort for a section, *R R*, like a common beam, which greatly simplifies the computations and errs on the safe side. Engineers are accustomed to making a similar assumption in case of concrete arches, when they determine the stresses in a section at right angle to the center line of the arch. In view of this fact, it would be well to calculate the fillet in Fig. 4 along a line about bisecting the angle, *N′ N I*, which will be found the dangerous section.

Mr. Nishkian.

L. H. NISHKIAN, ASSOC. M. AM. SOC. C. E. (by letter).—On page 761 the author computes the shear in section *N I*, Fig. 4. An important function of the inclined bar is to relieve the shearing stresses in section *N I*, but the author does not take into consideration the vertical component of the stress in the inclined bar. In order to have equilibrium of all the vertical forces acting on that portion of the base to the right section *N I*, the vertical component of the stress at section *N I* in the inclined bar should be deducted from the total vertical load, 18 333 lb., and the remainder will be the shear in the concrete in section *N I*.

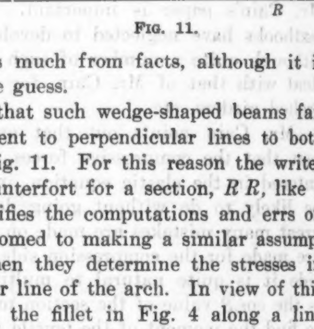

FIG. 12.

* "The Reinforced Concrete Pocket Book."

The sum of the vertical components, per foot of width of the stresses in the inclined bar, is Mr. Nishkian.

$$17\,750 \times 0.765 \times \frac{3}{2} \times \sin.\ 43°\ 10' = 14\,000 \text{ lb.}$$

and $$18\,333 - 14\,000 = 4\,333 \text{ lb.}$$

Therefore the maximum shear in section $N\,I$ is $47.6 \times \dfrac{4\,333}{18\,333} = 11.2$ lb. per sq. in. instead of 47.6 lb. per sq. in.

WILLIAM CAIN, M. AM. SOC. C. E. (by letter).—The writer would like to make a correction as to the meaning of v in Formulas (17) and (18), as others besides Mr. Janni may inquire the reason for the statement that "The unit stress, v, acts parallel to $N\ N'$". The statement is incorrect; v, in Formulas (17) and (18), is the unit shear on any plane, lying between O and the steel bars, perpendicular to $N\ I$. Mr. Cain.

Thus, suppose such a plane drawn in Fig. 8 and marked $A\ A'$ (A lying in $N\ I$, A' in $N'\ I'$), then since the section, $N\ I$, was taken parallel to the direction of the loads, at a point, A, where the bending stress is supposed to be zero, there is shear only on the plane, $N\ I$, and consequently a shear of equal intensity, v, on a plane, $A\ A'$, at right angles to $I\ N$, by a well-known theorem. The total shear on the plane, $A\ A' = v\ b\ dx$.

For equilibrium, the algebraic sum of the components perpendicular to $I\ N$, of the forces acting on $N\ A\ A'\ N'$ must be zero. Therefore

$$(C' - C)\cos.\beta = v\ b\ dx,$$

which is the same as the equation just above (17). Hence, substituting this value in the equation derived just before the last, or in,

$$(C' - C)\cos.\beta\ D\ G = V'\ dx,$$

dividing by dx and then taking the limit, as dx approaches zero and V' approaches V indefinitely, it is found that,

$$v = \frac{V}{b\ D\ G}$$

just as given in Formula (17).

Thus Formulas (17) and (18) are unchanged in form, but v is now to be interpreted to mean the unit shear on a plane perpendicular to $N\ I$, lying between the neutral axis and the steel.

This is the maximum shear, being greater than the shear on any other plane than $A\ A'$ passing through A; hence it is of special interest to the designer.

Mr. Janni makes a perplexing statement when he says: "cos. $(\alpha + \beta)$ is purely a symbol of a certain operation to be performed, and is not an actual value; its value is given by cos. α cos. β — sin. α sin. β." Perhaps this is a "*lapsus calami*". All our textbooks state that the cosine of any angle, as $(\alpha + \beta)$, is an actual value, and gives tables of the actual

Mr. Cain. values for various values of $(\alpha + \beta)$. Also, such values are often represented by certain lines on a unit circle, by aid of which the formula quoted is derived. By a consideration of such line values, it is at once seen that cos. $(\alpha + \beta)$ approaches indefinitely cos. β as α tends indefinitely toward zero, and the same result follows from using the development of cos. $(\alpha + \beta)$.

In fact, take the equation,

$$P\,Q = \frac{v\,\alpha}{\cos.\,\beta - \alpha \sin.\,\beta},$$

which Mr. Janni gives. As he knows, α sin. β is an infinitesimal, and can be neglected in comparison with the finite quantity, cos. β, when connected with it with either a plus or minus sign. Thus, as α tends toward zero, without ever becoming zero, the value of $P\,Q$ approaches $\frac{v\,\alpha}{\cos.\,\beta} = v\,\alpha$ sec. β, which is exactly the formula derived by the writer.

The writer cannot agree with Mr. Janni that the direction of the bending stress on the section, $N\ I$, varies in the manner shown in Fig. 9. It will presently be proved that the stress on the cross-section of an actual beam varies continuously in amount and direction, so that no sudden change in direction of 90° at any point can occur.

This is true even at points of contact of wheel loads, for such loads are in reality distributed. So far as dams are concerned, the changes in the amount and direction of the principal stresses along horizontal sections have been determined, both theoretically and experimentally.

Reference has been made to the experiments on india rubber model dams by Messrs. Wilson and Gore. Fig. 13 is taken from their paper,* giving an account of these very interesting experiments. The unit stresses, in amount and direction, at numerous points in the dam, were determined from the distortions, and are represented by the ellipses of stress shown in the figure. The principal stresses, or those acting normally to their planes, are represented, in amount and direction, by the semi-axes of the ellipses. Of these, the maximum normal stresses are given by the semi-major axes and are those that specially interest us.

It will be observed that along any horizontal section, these unit principal stresses change gradually, in amount and direction. They are also observed to act parallel to the inclined down-stream face, very near that face. At the up-stream face, where the dam is subjected to water pressure, the direction of the principal stress is nearly normal to that face, to counteract the large normal pressure of the water.

The same general results were first found theoretically by Mr. Ernest Prescot Hill,† for a dam with the up-stream face vertical, and later

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, Session 1907-08, Part II.

† In a paper on "Stresses in Masonry Dams," *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, p. 134.

by the writer* for the up-stream face battered; both writers adopting the usual approximate trapezoid law for the distribution of the vertical components of stress on a horizontal section. Mr. Cain.

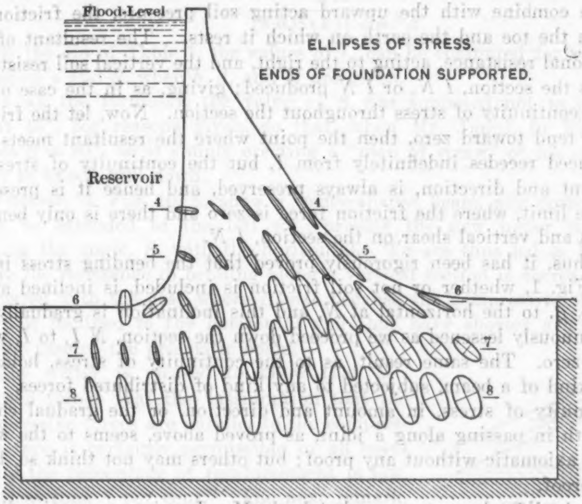

FIG. 13.

Fig. 14 represents the changes in direction of the greater principal stresses, as computed by the writer on a horizontal section of a dam, 200 ft. below the water level, the reservoir being full. For reservoir empty, the principal stress at the up-stream face would act parallel to that face.

Now, suppose that, for a dam of the same size and shape, but of less specific gravity, and capable of exerting tension, the resultant on the horizontal section passes much nearer the outer toe or beyond it; then a part of the section would be in tension, but the theory quoted shows that the continuity of stress, in amount and direction, would be preserved, and that there would be no abrupt changes in either, in going from the up-stream to the down-stream face. This continuity is preserved, as the center of pressure on the base moves indefinitely beyond the outer toe, and hence it is true at the limit, when the weight of the

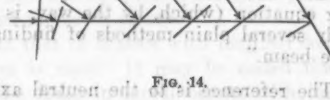

FIG. 14.

* *Transactions*, Am. Soc. C. E., Vol. LXIV, p. 208.

Mr. Cain. dam is neglected and the horizontal section is subjected to a pure bending stress and shear, due to the water pressure alone. Exactly the same reasoning applies to the toe, Fig. 1, supposed to be homogeneous, if we combine with the upward acting soil pressure, the friction between the toe and the earth on which it rests. The resultant of this frictional resistance, acting to the right, and the vertical soil resistance, meets the section, *I N*, or *I N* produced; giving, as in the case of the dam, continuity of stress throughout the section. Now, let the friction force tend toward zero, then the point where the resultant meets *I N* produced recedes indefinitely from *I*, but the continuity of stress, in amount and direction, is always preserved, and hence it is preserved at the limit, where the friction force is zero and there is only bending stress and vertical shear on the section, *I N*.

Thus, it has been rigorously proved that the bending stress in the toe, Fig. 1, whether or not soil friction is included, is inclined at the angle, β, to the horizontal at *N*, and this inclination is gradually and continuously lessened as we proceed down the section, *N I*, to *I*, where it is zero. The same result, as to the continuity of stress, holds for any kind of a beam, subjected to any kind of distributed forces. This continuity of stress, in amount and direction, or the gradual change in both in passing along a joint, as proved above, seems to the writer to be axiomatic without any proof; but others may not think so, hence the proof.

It is difficult to see on what basis Mr. Janni assumes the discontinuity of 90° in the direction of the stress in Fig. 9. Presumably, the stresses act in the direction of the dotted lines. An obvious remark is, that if such directions were the true ones, then the arms of the corresponding stresses are less than those corresponding to the writer's hypothesis; so that even the writer's formulas would be on the danger side for the distribution of stress shown in Fig. 9; and the ordinary formulas, which Mr. Janni advocates, would be still farther on the danger side. There is thus an inconsistency which is fatal to Mr. Janni's contention.

The writer is satisfied that Mr. Janni has not read carefully the derivations of his equations or he would not have stated with reference to Equation 3:

"It is not clear to the writer, however, why the author gives a rather long equation (which, by the way, is not correct), when there are already several plain methods of finding that axis in a reinforced concrete beam."

The reference is to the neutral axis. It is hardly necessary to remark that if the "plain methods" are those pertaining to a prismatic beam (the only case ordinarily treated), they are entirely inapplicable to a wedge-shaped beam.

From overlooking this fact, Mr. Janni's Equation 23 is in error when β is not zero. The correct formula, assuming parallel compressive stresses, is given by Equation 15. Mr. Cain.

$$f_c = \frac{2\,M}{(b\,d)^2\,(k\,j)\,\cos.^2\beta}$$

By the aid of Table 1, the values of f_c can be computed for various values of p and β, when b, d, and M are given.

In all the formulas, the value of β was supposed not to exceed about 45°, for reasons given, so that, when Mr. Janni, in Fig. 10, makes $\beta = 90°$, he is simply erecting a "man of straw" of his own in order to knock him down.

Further, in Fig. 10, to show a wedge-shaped beam, any dotted line must be extended to the left to represent the new upper surface, making the angle, β, with the horizontal. We are thus led to Fig. 1, where the resistance to bending along a vertical section is decreased, as compared with the case where $\beta = 0$, because the compressive stresses are now inclined to the horizontal and thus have shorter arms than for the case of the prismatic beam.

The writer is pleased to know that Mr. Mensch thinks that the "assumption that the compressive forces are parallel to the compression face is certainly permissible in all cases where the percentage of reinforcement is low, say, less than 1 per cent." The next remark, that it "errs only slightly on the side of danger", presumably refers to the hypothesis of conservation of plane sections being adopted, as in the usual theory. Some experiments of A. N. Talbot, M. Am. Soc. C. E., seem to substantiate the hypothesis, those of Professor Schüle going the other way. For working loads, the hypothesis is universally used, as it is found to give safe results for reinforced prismatic beams, hence its adoption by the writer for the wedge-shaped beam, with the distinct proviso, however, that it was not to be used for large values of β.

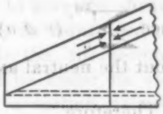

FIG. 15.

Not only on this account, but principally because it was thought that the hypothesis of the compressive stresses, all acting parallel to the face in compression, departed too much from the truth for large angles, β was limited to about 45°, though, possibly, the limit might be extended to $\beta = 60°$, if the results are tempered with good judgment, which means a slightly increased factor of safety.

Mr. Mensch refers to the fact that in trusses with inclined compression members, the right section is used. It may be added that the stresses are taken parallel to the faces in metal beams with inclined flanges. Suppose a reinforced concrete T-beam, Fig. 15, with an inclined flange; would not a designer naturally take the compressive stresses in the flange as acting parallel to its surface? It seems absurd

Mr. Cain. to take them as horizontal, particularly for the part of the flange on each side of the stem.

It is but one step now to the beam, Fig. 1, without a flange.

Mr. Mensch alludes to the fact that errors are sometimes made by using the factor, cos. β, in place of cos.^{2} β, in finding the compressive stresses. To emphasize this point, it should be remembered that A represents the area of a steel bar, taken at right angles to its axis, so that if f_s is the unit stress, the total stress in the bar is $f_s\ A$; but, considering a cylindrical "fiber" of the concrete, $N\ N'$, Fig. 2, the area of its section made by the plane, $I\ N$, is called Δa, so that the area of a section at right angles to the axis of the fiber is (Δa cos. β), and the stress on the fiber is thus, $f_c\ .\ \Delta\ \alpha$ cos. β, f_c being the unit stress on the normal section.

For the fiber, $P\ P'$, Fig. 2, the stress is, similarly, $\frac{f_c}{kd_1}\ v\ .\ \Delta\ a$ cos. β, where $O\ P = v$, and the sum of such stresses over the area, $O\ N$, is,

$$C = \frac{f_c}{kd_1} \cos.\ \beta \sum_0^{kd_1} (v\ \Delta\ a) = \frac{f_c}{kd_1} \cos.\ \beta\ \frac{1}{2}\ b\ (kd_1)^2,$$

since $\sum_0^{kd_1} (v\ \Delta\ a)$ is the statical moment of the area under compression about the neutral axis.

Therefore $$C = \frac{1}{2}\ f_c\ b\ .\ kd_1 \cos.\ \beta,$$

and the moment of C about G is,

$$M_c = \frac{1}{2}\ f_c\ b\ .\ k d_1 \cos.^2 \beta\ .\ D\ G \qquad (8)$$

For the special case given by Fig. 7, G coincides with I, and $D\ G = jd$, on putting $d_1 = d$. Therefore,

$$M_c = \frac{1}{2}\ f_c\ (kj)\ bd^2 \cos.^2 \beta = \frac{1}{2} f_c\ (kj) b\ p'^2,$$

where $p' = d$ cos. β = the perpendicular from I on $N\ N'$ produced.

Mr. Mensch states that he has advised calculating a wedge-shaped beam, Fig. 11, for a section, $R\ R$. It is not clear exactly what Mr. Mensch means by the statement, for there are mathematical difficulties in the way where the section, $R\ R$, is not parallel to the line of action of the loads. However, accepting the hypothesis of parallel compressive stresses, the last equation gives the precise result,

$$M_d = \frac{1}{2} f_e\ (kj)\ b\ .\ \overline{R\ R}^2$$

as $p' = R\ R$, of Fig. 11.

It is to be observed, however, that k and $j = 1 - \frac{1}{3} k$, must be determined for a section (say a vertical section in Fig. 11) through the lower R, parallel to the loads and not for the section, R R, as follows from the derivation of the formula. Mr. Cain.

Formula 16 is in the simplest possible form, and by aid of Fig. 6, the values of k can be read nearly to thousandths and the computation easily effected. As a matter of fact, the steel percentage for the toes, heels, or counterforts of retaining walls, is nearly always very low, so that the steel is the determining factor. In any case, both M_s and M_c should be computed from Equations 14 and 16, for assumed values of f_s and f_c, and the least of the two equated to M, the moment of the external forces; or, if possible, the steel section should be revised, because under-reinforcement is in most cases preferable to over-reinforcement, as it leads to a more progressive, and not a sudden, failure, in case of excess loading.

Mr. Nishkian raises an interesting point in connection with Fig. 4 (referring to the case of the heel-slab with the fillet), that the vertical component of the stress in the inclined rods, for the width, b, should be subtracted from the total external shear to give the shear in the concrete in section, I N. It may be observed, in reinforced concrete beams, that the bending of the beam entails the interaction of steel and concrete, without which the steel would be under no stress whatever. Thus the tensile stress in any bar, at a section, is necessarily accompanied with a reaction or an equal bond shearing stress in the concrete, along the entire surface of contact of the bar, to one side of the section. The steel elongates under stress, and this elongation is resisted by the concrete and the total reaction, for the length of bar to one side of the section—the total bond stress—is exactly equal to the tension in the bar. Thus the stress in the bar does not relieve the concrete of shear directly. Even where there are hair cracks on the tensile side of the beam, the full bond stress is exerted between the cracks, giving rise to ordinary shears, and these in turn to indirect tensions and compressions in the parts intact, which play an indispensable rôle in transmitting stresses and maintaining the integrity of the beam.

To deduce a formula for shear, only a part of the beam, such as is represented in Fig. 8, is considered. In an actual beam, as repeatedly pointed out, the compressive stresses are not all parallel to the face in compression, but their inclinations to the normal to I N become less as the neutral axis is approached. In Fig. 8, the hypothesis of parallel compressive stresses is only used to establish approximately the point, D, the point where the resultant, C, of the compressive stresses cuts I N. If C is now thought of as the resultant of the actual compressive stresses, having varying inclinations to I N, and calling β' its inclination to the normal to I N, the formulas which lead to Equation 17 are unchanged,

Mr. Cain. provided we substitute β' for β. The angle does not appear in the final result. To realize further the conditions in an actual beam, tensile stresses in the concrete must be supposed between the neutral axis and the point where the hair cracks are experienced. Although, in the ordinary theory, such tensile stresses are ignored, yet it is seen, because they actually exist, that the section for maximum shear should strictly be taken at O, or at the neutral axis. Such ignored tensile stresses, though small, are nevertheless effective in changing the directions of the stresses in the concrete between the neutral axis and the point where cracks appear.

Formula 17 can be deduced in a different way from that given, which will introduce that interaction between the steel and concrete first alluded to. Let it be assumed that only shear of intensity, v, is exerted on a plane just below O, Fig. 8, that is, perpendicular to $I\ N$. Hence, adopting the previous notation pertaining to Fig. 8, the total shear on the plane is $v\ .\ b\ .\ dx$.

The bond stress exerted at the surface of contact of any bar, as $I\ I'$, throughout the length, $I\ I'$, is equal to the difference in the tensile stresses in the bar at I and I', and it acts in the direction, $I'\ I$. Hence the total bond stress exerted by all the bars is $(T' - T)$ in amount, and it acts in the opposite direction to T', if $T' > T$.

If we call β_0 the angle made by the resultant, T (or T'), with the normal to $I\ N$, then $(T' - T) \cos. \beta_0 =$ the horizontal component of the total bond shearing stress exerted by all the bars, and this must equal the shearing stress, $v\ .\ b\ .\ dx$, exerted on the plane perpendicular to $I\ N$, just below O. Therefore,

$$v\ .\ b\ .\ dx = (T' - T) \cos. \beta_0.$$

Next, considering the free body, $N\ I\ I'\ N'$, held in equilibrium by the forces, V, V', C, C', T, T', acting as shown in Fig. 8, and taking moments about D,

$$(T' - T) \cos. \beta_0\ .\ \overline{D\ G} = V'\ .\ dx.$$

Eliminating $(T' - T) \cos. \beta_0$ between these two equations, dividing by dx, taking the limit as dx tends toward zero and V' tends toward V, and solving for v, we derive,

$$v = \frac{V}{b\ .\ \overline{D\ G}},$$

or Equation 17, which was first derived by a slightly different method.

At first sight, it might appear that the plane just below O, on which there is only shear, should be parallel to $T\ T'$, so that the shear on it should exactly equal the bond stress $(T' - T)$, which must be transmitted by ordinary shear to the neutral axis; but when we consider that along $O\ S$ (by the hypothesis), there is no bending stress, and

hence no component of bending stress normal to $O\ S$, it follows that there can be only shear on $O\ S$, and hence a shear only of equal intensity on a plane normal to $I\ N$, say just below O, or between the neutral axis and the steel, if we ignore all tensile stresses in the concrete. The first equation above is thus justified. In the final equation for v, the whole external shear, V, appears, and there seems to be no justification for diminishing it by the component parallel to $I\ N$ of the pull in the rods. Mr. Cain.

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Paper No. 1296

A STUDY OF ECONOMIC CONDUIT LOCATION.

By C. E. Hickok, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. H. Hawgood, M. F. Stein, and C. E. Hickok.

The paper entitled "Economic Canal Location in Uniform Countries,"* by Lyman E. Bishop, Assoc. M. Am. Soc. C. E., contains a series of interesting and useful diagrams, by the use of which the locating engineer can quickly determine the economic center line cut for any particular canal section for any slope of ground. However, every conduit, unless it is in a country of uniform topography, must change at certain points from one type of construction to another, in order to be built economically and safely. It is rarely that a conduit of any considerable length can consist entirely of canal section, but rather it must change to flumes, siphons, pipes, bridge flumes, or tunnels, as the conditions demand. The points of change are determined, not only by the slope of the ground, the nature of the material encountered, and certain local conditions, but by economic considerations as well.

In making conduit locations, from time to time, the writer has evolved a diagram, giving the equivalent lengths, from an economic standpoint, of various types of conduit, which has been of considerable value. For instance, when the locator comes to a point where he must decide whether to tunnel through a ridge or follow the grade around with a canal, he measures the length of the two possible routes, and, by an inspection of the diagram, comes to a ready decision. This not only eliminates considerable loss of time, but, if the diagram has been properly constructed, assures a proper and complete comparison between

* *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 179.

the two alternatives as to first cost, depreciation, head-loss values, evaporation and seepage loss values, interest, taxes, inspection, and repairs.

For purposes of illustration assume a case where the project under consideration is to be used for irrigation and hydro-electric purposes, and where the conduit has a capacity of 44.6 cu. ft. per sec. and a slope of one-tenth of 1 per cent. Four types of conduit are shown in Fig. 1.

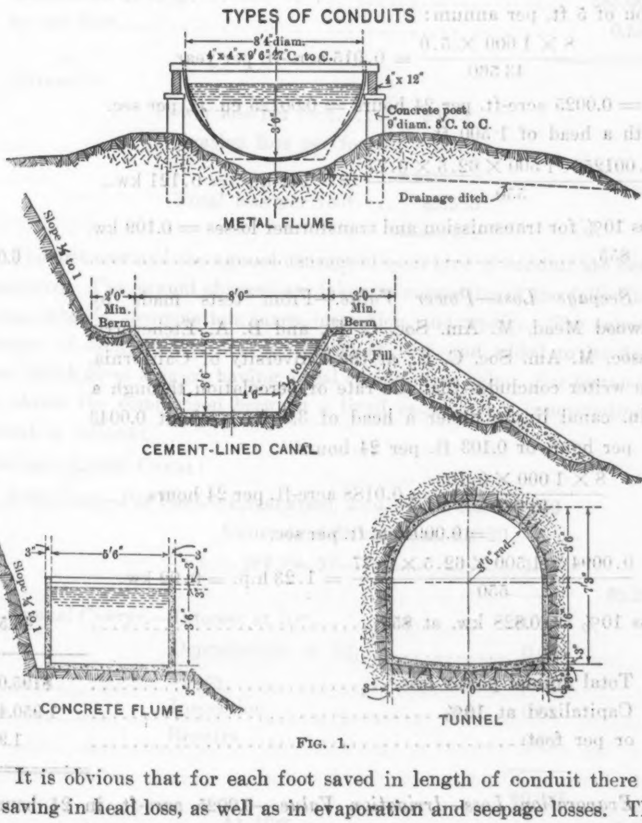

FIG. 1.

It is obvious that for each foot saved in length of conduit there is a saving in head loss, as well as in evaporation and seepage losses. The value of this saving is ascertained in the following way, taking 1 000 ft. of conduit, for convenience in calculating:

Head Loss.—1 000 ft. of conduit dissipates 1 ft. head. With a discharge of 44.6 cu. ft. per sec., and 77% efficiency, the horse-power is

$$\frac{1 \times 44.6 \times 62.5 \times 0.77}{550} = 3.9 \text{ h.p.} = 2.8 \text{ kw.},$$

less 10% for transmission and transformer losses = 2.61 kw. at \$55 .... \$143.50

Evaporation Loss—Power Value.—Assuming an evaporation of 5 ft. per annum:

$$\frac{8 \times 1\,000 \times 5.0}{43\,560} = 0.915 \text{ acre-ft. per year}$$

$= 0.0025$ acre-ft. per 24 hours $= 0.00125$ cu. ft. per sec. with a head of 1 500 ft.,

$$\frac{0.00125 \times 1\,500 \times 62.5 \times 0.77}{550} = 0.162 \text{ h.p.} = 0.121 \text{ kw.},$$

less 10% for transmission and transformer losses = 0.109 kw. at \$55 .... 6.00

Seepage Loss—Power Value.—From tests made by Elwood Mead, M. Am. Soc. C. E., and B. A. Etcheverry, Assoc. M. Am. Soc. C. E., at the University of California, the writer concludes that the rate of percolation through a 3-in. canal lining under a head of 3.5 ft. is about 0.0043 ft. per hour, or 0.103 ft. per 24 hours.

$$\frac{8 \times 1\,000 \times 0.103}{43\,560} = 0.0188 \text{ acre-ft. per 24 hours}$$

$$= 0.0094 \text{ cu. ft. per sec.}$$

$$\frac{0.0094 \times 1\,500 \times 62.5 \times 0.77}{550} = 1.23 \text{ h.p.} = 0.92 \text{ kw.}$$

less 10% = 0.828 kw. at \$55 .... 45.54

Total annual power loss	\$195.04
Capitalized at 10%	1 950.40
or per foot	1.95

Evaporation Loss—Irrigation Value.—0.0025 acre-ft. in 24 hours (from the foregoing) = 0.00125 cu. ft. per sec. = 0.0625 miner's inch. Assume 25% loss before delivery to consumer:

0.047 miner's inch at \$0.40 per miner's inch per day = per annum \$6.86

Seepage Loss—Irrigation Value.—0.0188 acre-ft. per 24 hours (from the foregoing) = 0.0094 cu. ft. per sec. = 0.47 miner's inch, less 25% loss = 0.353 miner's inch at \$0.40 per miner's inch per day = per annum 51.64

Total annual irrigation loss	\$58.50
Capitalized at 10%	585.00
or per foot	0.585

Résumé.—

Power loss per foot	\$1.95
Irrigation loss per foot	0.585
Total loss per foot	\$2.535

The first cost and the annual charges of each type of conduit are next computed. The annual charges are taken as consisting of the following items: interest, depreciation, taxes, inspection, and repairs. The annual charges of each conduit are capitalized at 10% and added to its first cost, which gives a figure having a real comparative value. For instance, we obtain the comparison between a lined canal and a concrete-lined tunnel as follows:

CONCRETE-LINED CANAL:

First Cost—Per Foot.—	Excavation, 2 cu. yd. at \$0.36	\$0.72	
	Concrete, 4.25 cu. ft. at \$10.20 per cu. yd.	1.57	
			\$2.29
Annual Charge.—	Interest at 10%	\$0.23	
	Depreciation at 2%	0.046	
	Taxes	0.019	
	Inspection	0.01	
	Repairs	0.02	
		\$0.325	
	At 10%		3.25
			\$5.54

CONCRETE-LINED TUNNEL:

Excavation, 2.25 cu. yd. at $5.50	$12.40	
Concrete and forms	4.10	
		$16.50

Annual Charge.—Interest at 10%	$1.65		
Depreciation at 1%	0.165		
Taxes	0.137		
Inspection	0.01		
Repairs	0.02		
		$1.982	
At 10%			19.82
			$36.32

It is evident, if we shorten the conduit by building the tunnel, that the first cost and the capitalized annual cost of the tunnel can exceed the first cost and the capitalized annual cost of the canal by an amount equal to the length of conduit saved multiplied by the loss value per foot of conduit. This is shown by the equation:

$$Y\ (C_y + A_y) = X\ (C_x + A_x) + (x - y)\ V$$

where X = linear feet of canal,
Y = linear feet of tunnel,
C_x = estimated cost per foot of canal,
A_x = estimated annual charges per foot of canal capitalized at 10%,
C_y = estimated cost per foot of tunnel,
A_y = estimated annual charges per foot of tunnel capitalized at 10%,
and V = value of losses per foot of conduit.

In the case of a tunnel, the evaporation will be considerably lessened, thereby effecting an additional saving. If entirely eliminated, this saving would amount to 12.8 cents per ft., as shown above. This was reduced to 10 cents and the first cost of tunnel credited with that amount. Inserting the proper values in the equation:

$$Y\ (16.40 + 19.82) = X\ (2.29 + 3.25) + (x - y)\ 2.53$$

$$Y = 0.208\ X, \text{ the equation of a straight line.}$$

In the same way, any two types of conduit can be compared and the resulting straight-line equation obtained. The diagram, Fig. 2, which is self-explanatory, shows the results.

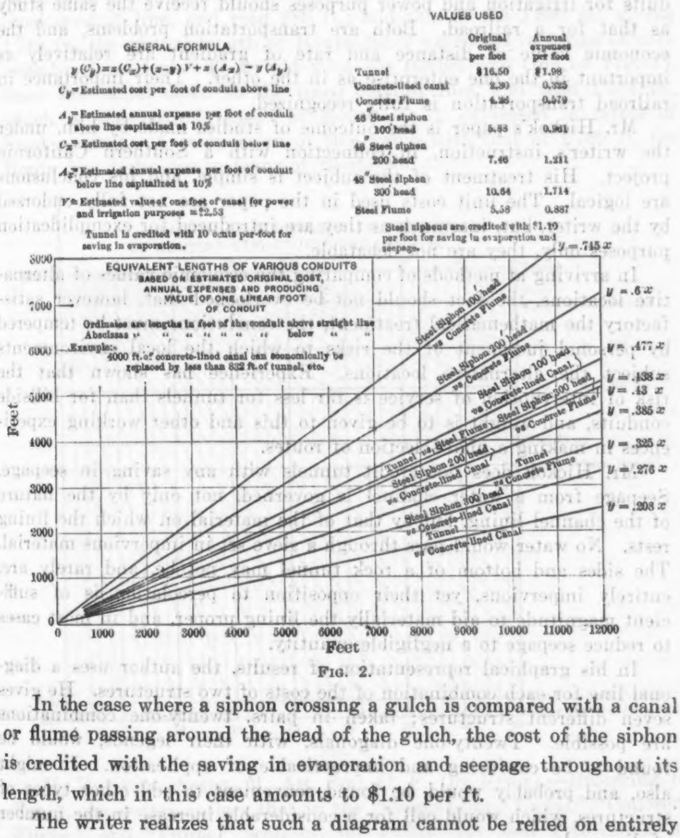

FIG. 2.

In the case where a siphon crossing a gulch is compared with a canal or flume passing around the head of the gulch, the cost of the siphon is credited with the saving in evaporation and seepage throughout its length, which in this case amounts to $1.10 per ft.

The writer realizes that such a diagram cannot be relied on entirely in the location of a conduit, for there are local conditions on every piece of work which must be taken into account.

DISCUSSION

Mr. Hawgood.

H. Hawgood, M. Am. Soc. C. E. (by letter).—The location of conduits for irrigation and power purposes should receive the same study as that for a railroad. Both are transportation problems, and the economic value of distance and rate of gradient are relatively as important in the one enterprise as in the other. Their importance in railroad transportation is fully recognized.

Mr. Hickok's paper is the outcome of studies made by him, under the writer's instruction, in connection with a Southern California project. His treatment of the subject is simple, and his conclusions are logical. The unit costs used in the paper are not fully endorsed by the writer, but, inasmuch as they are introduced for exemplification purposes only, they are not debatable.

In arriving at methods of comparing the economic values of alternative locations, the fact should not be overlooked that, however satisfactory the mathematical treatment, the conclusions must be tempered by personal judgment of the risks to which the local environments subject the alternative locations. Experience has shown that the risk of interruption of service is far less for tunnels than for hillside conduits, and weight is to be given to this and other working experiences in making a final selection of routes.

Mr. Hickok does not credit tunnels with any saving in seepage. Seepage from a water channel is governed, not only by the nature of the channel lining, but by that of the material on which the lining rests. No water would pass through a sieve set in impervious material. The sides and bottom of a rock tunnel may not be, and rarely are, entirely impervious, yet their opposition to percolation is of sufficient magnitude to aid materially the lining proper, and in most cases to reduce seepage to a negligible quantity.

In his graphical representation of results, the author uses a diagonal line for each combination of the costs of two structures. He gives seven different structures; taken in pairs, twenty-one combinations are possible. Twenty-one diagonals, with their legends, would be found to be confusing and impracticable of application. It might also, and probably would, be found convenient to add other types of structures, which would call for a considerable increase in the number of diagonals.

The writer suggests Fig. 3 as a more convenient form of diagram and as covering all phases of the subject. The radiating lines permit finding the equivalent distance, or type of structure, in terms of length of any other type of structure. The horizontal lines, each representing a particular type of structure, are spaced so that their distances apart vertically bear the same ratio one to the other as their respective unit costs. To exemplify the use of the diagram, assume that, of two

alternative routes, Route No. 1 has 200 ft. more tunnel and Route No. 2 has 1 500 ft. more lined conduit and 300 ft. greater length of line. Find Vertical 3 on the base line representing the economic value of 1 ft. of distance; trace the radial thence up to the concrete-lined conduit line, and read 289 ft.; add this to the 1 500 ft., giving 1 789 ft., the virtual length of Route No. 2 expressed in terms of concrete-lined conduit. Then find 2 on the tunnel line (top of Fig. 3), trace the radial down to the lined conduit line, and read 1 324 ft., which is the virtual length of Route No. 2 expressed in terms of lined conduit. A comparison of this value with that of 1 789 ft. for Route No. 2, decides in favor of Route No. 1. Mr. Hawgood.

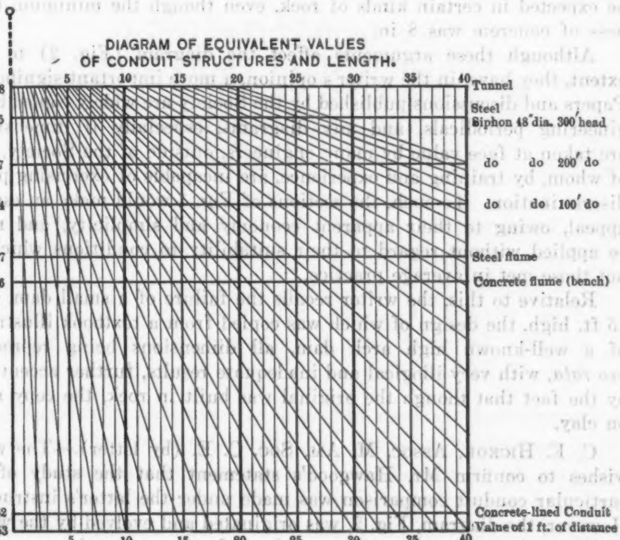

FIG. 3.

M. F. STEIN, ASSOC. M. AM. SOC. C. E. (by letter).—Referring to Fig. 1, the writer notices that in the "Cement-lined Canal," "Concrete Flume," and "Tunnel" sections, the author has used a thickness of concrete of 3 in. It would seem, from the writer's experience and observation, that though this thickness may be theoretically sufficient to withstand the water pressure, it may not give a sufficient factor of safety for various contingencies which cannot be calculated. It is generally difficult to obtain water-tight construction with a thin wall, using the customary type of labor and superintendence in mixing and placing the concrete, and frost action may prove destructive in Mr. Stein.

Mr. Stein.

a cold climate, especially in the event of seepage through the concrete. A thin section, furthermore, provides little safety against cracking, should there be any unequal settlement. These points would apply with less force in a warm and arid region, but the author does not limit his discussion to such conditions.

As regards the tunnel lining, though this may be sufficient for very firm rock, where it is only desirable to obtain a smooth surface, for hydraulic reasons, it would not offer much resistance to the rock movements, shelling, and thrusts experienced in limestone, shale, or clay formations. This is exemplified in the recent failure of the Montreal conduit, under an apparent pressure much less than might be expected in certain kinds of rock, even though the minimum thickness of concrete was 8 in.

Although these arguments affect the diagram (Fig. 2) to some extent, they have, in the writer's opinion, a more important significance. Papers and discussions published by the Society are widely copied by engineering periodicals, and any diagrams, especially if dimensioned, are taken at face value by many engineers, outside of the Society, some of whom, by training and experience, are incapable of exercising proper discrimination. To such, the sections of Fig. 1 would make an especial appeal, owing to their apparent economy and simplicity, and might be applied without regard to their suitability to conditions which are not those met in average practice.

Relative to this, the writer recalls the failure of a small dam, about 15 ft. high, the design of which was copied from a textbook illustration of a well-known high arch dam, all dimensions being reproduced *pro rata,* with very illogical and inadequate results, further accentuated by the fact that though the original was built in rock, the copy rested on clay.

Mr. Hickok.

C. E. Hickok, Assoc. M. Am. Soc. C. E. (by letter).—The writer wishes to confirm Mr. Hawgood's statement that the study of this particular conduit comparison was made under the latter's instruction. However, the diagram, Fig. 2, was originated and evolved by the writer.

The writer did not credit tunnels with any saving in seepage, as he was unable to ascertain even an approximate figure for this saving. He agrees with Mr. Hawgood that there would be a saving, but this depends so much on the nature of the tunnel material, which may vary from a pervious soil to hard rock, that there was a hesitancy as to what value to place thereon. There might even be an addition to the discharge of the conduit in the tunnel due to underground waters.

Mr. Hawgood's diagram, Fig. 3, is interesting and is certainly more compact than that of the writer.

In regard to Mr. Stein's remarks as to the thickness of the concrete in the different types of conduit, it should be stated that these drawings, Fig. 1, were not inserted in the paper to illustrate construction details,

but simply for pictorial purposes. The drawings were not necessary, and possibly should have been omitted. The writer hopes that no engineer would attempt to construct, for instance, a concrete flume similar to that shown on Fig. 1, without reinforcement. In regard to the 3-in. thickness of concrete lining in the tunnel, the section was supposed to be in solid rock, and no concrete lining was intended to be shown above the springing line. This escaped the writer's notice, due to the fact that he was more interested in the real subject than in construction details. Mr. Hickok.

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Paper No. 1297

THE DEPRECIATION OF PUBLIC UTILITY PROPERTIES AS AFFECTING THEIR VALUATION AND FAIR RETURN.*

BY JOHN W. ALVORD, M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. W. J. WILGUS, J. E. WILLOUGHBY, FRANK C. BOES, ALLEN HAZEN, H. C. VENSANO, LEONARD METCALF, WILLIAM B. JACKSON, F. LAVIS, ALEXANDER C. HUMPHREYS, HENRY FLOY, CLINTON S. BURNS, STUART K. KNOX, J. H. GANDOLFO, CHARLES RUFUS HARTE, W. KIERSTED, GEORGE B. STONE, AND JOHN W. ALVORD.

Depreciation has been discussed so fully since the regulation of public properties has become important, that space will not be taken here to review fundamentals. This discussion is directed to the question of the general relation of depreciation and its effects on the fair return of a utility property, and also, to some extent, to methods of accounting for depreciation in the administration of such properties.

For the general purpose of this paper, it will be considered that the term "depreciation" covers all the losses of value that occur in property, plants, or parts thereof, from wear and tear, obsolescence, or inadequacy. In other words, it will make no difference in this study whether the loss of value arises because the structure is outgrown, becomes obsolete through changes in the art, or is merely worn out. All these various ways by which value is lessened will be classed under the general term, "depreciation".

* Presented at the meeting of December 17th, 1913.

The Reproduction Method of Valuation.—In dealing with depreciation as affecting valuations, it must be clearly borne in mind that we are discussing only that line of evidence as to value known as the reproduction method, or cost new less depreciation. This line of evidence is only one source of information as to value, because plants or properties may or may not be worth what it costs to reproduce them, and no appraisal or valuation is at all complete which relies wholly on reproduction or cost new less depreciation methods. Other lines of evidence are equally important before arriving at value.

The reproduction method, however, is an important line of evidence, and must always be studied; and the relation of depreciation to cost new is therefore vital. This relation is discussed herein.

Depreciation as Affecting the Fixing of Rates.—Recently, the question has been asked, "Should depreciation be deducted from the investment cost of a property before computing fair return?" This question was discussed by C. E. Grunsky, M. Am. Soc. C. E., in his paper entitled "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates."* Mr. Grunsky contends that depreciation should not be deducted for rate-making purposes, but the original investment should be used for the fixing of rates, on the theory that the investment must be maintained intact, first, by the maintenance of the plant, and, second, by a sum set aside for renewals from time to time. The question is also discussed in a paper by Mr. Robert H. Whitten,† of the New York Public Service Commission, who concludes that, for rate-making purposes, the depreciation should be deducted from the reproduction cost. Mr. Samuel S. Wyer,‡ of Columbus, Ohio, expresses the opinion that the cost of reproduction. less accrued depreciation, is not the proper basis for rate-making, but does not make it quite clear whether he means this to apply to cases where the sinking fund for depreciation is in hand or not.

Here, therefore, there seems to be a very serious disagreement, among authors who have written recently on this subject, as to whether or not depreciation should be deducted from the cost new of a property for finding value for rate-making purposes where the reproduction method is adopted as a basis of value. It seems to be desirable, there-

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 770.

† *Engineering News*, May 8th, 1913.

‡ In his book, published recently, "Regulation, Valuation, and Depreciation of Public Utilities," Chapter 13, p. 185.

fore, to present a further study of this question. Certainly there must be in this, as in all other questions relating to this difficult subject of valuation, a just and equitable relation between the public and the public utility owners, which is founded on rational analysis and common sense.

At the outset, it may be said that the sinking-fund method seems to be an accurate way to compute depreciation where all or most of the life history is known, as it consists largely in finding past depreciation. This is mainly because the sinking fund is without emolument to the owners of the plant, and therefore has in it no element of confusion.

The sinking-fund method of computing or allowing for depreciation consists in determining the proper useful life of the structure or machine under consideration, and setting aside from year to year a sum of money which, with its annual increment of interest, will, at the end of the assumed life, replace the structure or machine in question.

In case a portion of the life has passed away, the present value of such structure or machine is assumed to be, in a general way, the present cost of replacement new, less the accrued amount of the depreciation fund to date.

The sinking-fund method, however, when practically applied to a great variety of structures in a plant of a wide and varied character, is a complicated and difficult way for the accountant to write off depreciation for the future, especially where most of the data must be assumed and where they do not warrant that degree of accuracy that a sinking-fund computation would imply.

It is often found practical, therefore, and rational, to determine past depreciation on the terms of a sinking-fund basis, while future depreciation may be practically set aside on a uniform-increment method for simplification. There seems to be no reason why the two methods cannot be made to correspond, if desired, or why this combination is not as rational, under the circumstances; as the future is quite largely prophecy, in any event.

In dealing with theory, however, it seems best to use the sinking-fund method in illustration, as it appears to be the accurate way in which to think out the governing principles.

Actual versus Theoretical Depreciation Funds.—Let it be clearly understood, first of all, that a sinking fund for depreciation, actually

set aside under suitable trust provisions, is to be considered as a part of the property, and, where it is found intact and of proper amount by the appraiser, it should be thus considered, and no deductions from cost new or reproduction cost, where that method is being used, would be necessary.

Where a proper depreciation fund is not set aside actually, but is only estimated and written off in the accounts, there is no such condition, and the matter of writing it off by the owner in his books amounts to only an opinion of his as to what such depreciation fund should be, but does not at all produce such a fund, or make it available; in consequence of which, an appraiser, using the cost new or reproduction method as one of the means of arriving at the value of the property, and finding no depreciation fund actually in hand or in trust, is obliged to deduct it from the property.

In other words, no amount of accounting or theorizing can make good a depreciation fund which is actually not in hand. A clear perception of this principle will save much confusion of thought.

The Attempt to Simplify the Question.—In order to understand the dilemma which a certain line of thought has been recently attempting to follow in this matter, an effort will be made, first of all, to state the case from such a standpoint.

Probably the most simple proposition which would occur to one first considering this question would be the case where the composite life of an entire plant or property has been estimated; that is to say, where the depreciations of all its component parts have been thoughtfully determined, the probable life of each component part established, its age found, and the amount of a sinking fund which will replace it at the end of its life computed. The average life of these component parts should give the composite life of the plant, at the end of which—theoretically at least—it would be subject to entire renewal; in other words, at that time, supposedly, the original investment would have completely disappeared.

To state more fully this line of thought (which has led to certain perplexities), let it be assumed for the moment that no additions are made to the plant, and that it is the purpose to hold the original investment intact. It is commonly considered in such case that if one computes (but does not set aside) a sinking fund for renewals, its accumulations, year by year, if properly estimated, will represent in a general way the decreasing value of the property, or at least roughly

so. If one deducts this sinking fund each year from the original value, to determine the value on the reproduction basis, and computes the rates which would be necessary in order to pay all charges and a fair return on the same, one would have, theoretically at least, a slowly decreasing valuation, year by year, on which to predicate rates, and the rates themselves would therefore necessarily lessen year by year until at the end of the composite life of the plant, there would properly be no rates at all, and, theoretically at least, the entire investment would be represented in the sinking fund, if there was one.

In California, under the laws which formerly existed for some years, such conditions seemed to be contemplated, for it was expected that plants would be valued, or might be valued, every year, for rate-making purposes. It was the existence of this law that probably led Mr. Grunsky to speculate on this particular problem, and his paper has convinced some students of the subject that the rates should not decrease with the depreciated value of the plant from year to year, but should be predicated on what he calls the "original investment", without deducting depreciation. That this appears plausible on its face, one would not at first be inclined to doubt, but it is believed that closer analysis will show that it is not sound in reasoning.

It should be noted, of course, that a sinking fund itself actually set aside is not a source of revenue to the owner, because its interest accretions are necessary to its own growth and purpose; therefore, it is argued that it may properly receive return outside of its own interest accretions. At the end of the life of a given plant, under this theory, it will be receiving the full rates on the full investment, but with a practically worn-out plant, together with a sinking fund in the bank which will entirely replace it. Theoretically, also, it is argued that, if the owner were to go out of business, he could take the entire fund with him, and consider himself recouped for his property. Of course, he would not be entitled to rates any longer. If it is supposed that he should decide to remain in business, he would immediately have to invest the entire sinking fund again in complete renewals, and receive rates based on the full necessary investment, all of which seems to point superficially to the conclusion that the rates, at all times, should be determined by the value of the undepreciated property.

There is much more to the question, however, than the above simple presentation would seem to indicate.

Perpetual Life.—As a matter of fact, depreciation is not as simple in its practical operation as just stated. In the first place, it is not contended seriously by any one familiar with the subject that a plant usually or normally has a point at which its life actually terminates.

A casual inspection of the life of various structures used by appraisers shows that, in any given plant, a great variety of useful existence is assigned from experience to different structures, so that actually none of the component parts of a plant can be said to terminate at one and the same time, but each part must be renewed independently at such time as the necessity arises. This would result in practically repeated reductions of the depreciation fund, if one actually existed, and the absorption of its funds from time to time into the plant. Therefore, there is no one time at which the depreciation fund actually does or can rise to a very large amount; and it would be practically impossible, under normal conditions, that it should ever grow in amount, even to the larger part of the original investment. As a matter of practical observation, it rarely ever theoretically amounts at any one time to more than from 20 to 25% of the original investment, and more frequently is from 10 to 15%, because, as a matter of practical necessity, renewals must be made and the plant must be maintained and perpetuated, whether the owners would desire it or not. Commonly, therefore, one does not have the condition that the plant is wholly, or even largely, renewed by the depreciation fund at any one time, except under the abnormal circumstances of complete failure through errors in original judgment, or, in rare cases, by reason of duplication or other ignorant lack of conservation.

To illustrate: for instance, in the case of a water-works plant, the boilers, ordinarily, will need to be renewed about once in every 20 years. They would probably be renewed about five times in 100 years, and at the end of about 75 to 150 years the pipe system, theoretically, would be renewed to a considerable extent. The buildings, on the other hand, might be renewed about two and one-half or three times during this average interval, and reservoirs about one and one-half or two times, so that, at the end of 100 years, there might still be a plant of some kind, existing in much the same general

form as the original one, except that those parts which may have become obsolete through changes in the art or method would require that their renewals take other form and character. Therefore, one is dealing practically with a perpetual aggregation of structures, which, not only must be maintained, but must also grow, expand, and enlarge. It is true that in some instances public utility plants have been built, run for a time, and abandoned. This is less and less true as time goes on and such enterprises are better understood. These cases of abandonment are so few as practically to be negligible in reasoning out this matter. The tendency of the times is to conserve and protect such property more and more from destruction or failure.

Utilities Commonly Require Growing Plants.—As a matter of fact, in considering depreciation, the original investment must not only be continued intact by frequent renewals, but, in all ordinary growing cities, must be rapidly increased from time to time by the addition of new capital expended in extensions and enlargements. These new extensions, in turn, have their depreciation, which must be provided for, and they, in turn, must be renewed when their time of usefulness has expired. The actual depreciation, therefore, at any given date in any plant, is the sum of all the depreciations of the various structures then in place, duly considering their age and probable life.

Earning Power of Depreciated Plants.—In considering the matter of the depreciation of a public utility plant, one must not confuse its loss of physical value, as measured by the growth of the depreciation funds, with its ability to have earning power. The progress of a plant in earning power is almost always distinct from its progress in depreciation or appreciation, and though it is generally considered that depreciation of physical property is usually followed by lessened earnings, this is not always true. A plant may not be well maintained, but its earning power may at the same time be well maintained, or even grow rapidly. Theoretically, one may sometimes have the condition (in certain kinds of plants at least), as in the sample illustration used at first, where the depreciation, as measured by the deduction of a sinking fund, becomes greater and greater year by year when plant renewals are not properly made, and where an almost worn out plant may be able to earn much more than the return it earned when new.

Of course, there is a limit to this condition; with certain kinds of plants it is not usually possible; and the sin of too long deferred renewals often brings heavy punishment, yet the fact remains that the earning power of a plant is not always controlled by the defects of depreciation.

Natural Accretions.—A plant or property not only depreciates, but also has sources from which it may appreciate. These appreciations of value have been held to be part of the property, by the Supreme Court of the United States, in the Monongahela Navigation case, and more recently in the case of the Kings County Lighting Company *vs.* The Public Service Commission of New York, First District. One of the reasons the Courts have for favoring the reproduction method, or cost new less depreciation, lies in the fact that such procedure includes appreciation as well as excludes depreciation.

When dealing with the valuation of public utilities, it must be kept clearly in mind that a "property" is being considered, and that property may be defined as being all that can be properly transferred to a buyer. Pipes or conduits laid in crowded streets, where many other conduits and tracks, pavements, and heavy traffic would make it difficult to replace them, certainly have a greater reproduction value than they had originally, when the city was unimproved and the streets were open and free for their installation. Right of way and lands, of course, have natural accretions in value, and this is well recognized and understood by the public. What the public does not understand is that similar accretions in value accrue to the property in other items as well as in the increased value of the land.

The Constitution of the United States, in referring to that which must be protected from confiscation, uses the word "property", and not "investment", and the term "property" must necessarily include all the assets which inevitably inhere to, and can be transferred with, the whole entity used and useful for the public.

There are some increments of value produced in connection with an enterprise which originally entailed considerable investment, but cannot be thus transferred, and therefore do not form permanently a part of the property. For instance, a company may have expended a great deal of time, pains, and money, in its original introduction of light, or water, or transportation, for the education of the public as to the value that arises from the use of these facilities. Where

a whole community has been thus educated, it has acquired a fundamental conviction that a utility is really necessary and useful, which conviction, though perhaps developed largely at the expense of a utility company, cannot be transferred as an asset with its property. This is the reason that, in computing costs of development in a rebuilding estimate, it is usually assumed that the public is now already educated in the use of the facility, and would quickly avail itself of the privileges if such facility were reinstalled.

Sinking Funds an Inherent Part of the Plant.—If money has been invested in property which is to be maintained perpetually, and if the public is to be protected by requiring only a fair return based on reasonable rates for the use of this property, such property must be maintained intact by setting aside, as part of the operating expenses, amounts which will sufficiently renew it in part from time to time. In the Knoxville case, the Supreme Court of the United States has held that this is reasonable and proper. If, therefore, one has the right to set aside this money from time to time, theoretically, one should set aside just so much as will maintain the original property in its original form, or the property plus its additions, accretions, and betterments in their total form. To do this will necessitate having on hand, theoretically, part property and part renewal fund. The renewal fund, therefore, is just as much an inherent part of the property as are the tracks, stacks, or boilers, if the property is to be considered as held intact. Where, therefore, one has to value a plant for sale or transfer, and does not find that a renewal fund has been accumulated, or is actually on hand, it is right and proper that one should, so to speak, "fine" the owners, for the absence of this part of the property, by the amount thereof. If the sinking fund for renewal is in the bank or in trust, as well as accounted for on the books of the company, and inherently a part of the property, and in proper amount, one should, naturally, on the other hand, give to the owners of such property the entire reproduction cost of the property undiminished by any depreciation deduction whatever.

It is clear, therefore, that, as far as valuation for the purpose of sale is concerned, it is right and proper that, in the absence of actual sinking funds or any sufficient reserve fund, they should be deducted from reproduction cost or "cost new", when that method is used to

ascertain value. Does not this also hold good in the case of valuation for rate-making?

The Practical Treatment of Depreciation Accounts.—As a matter of fact, few, if any, administrators of public utilities have ever (as far as known, up to date) actually accumulated a sinking fund. Most companies otherwise invest money which would theoretically go to such a fund, because they have thought that such moneys could usually earn a greater rate of interest than they would in a guaranteed sinking fund, and there is the feeling, also, that there would be no difficulty or hazard in relying on the ability of the owner or owners to replace the sinking fund from outside sources from time to time as demands on it occur. This is especially true where the management is more or less personal, and is perhaps not improper, from the point of view of a stockholder, but it means, however, that at all times the property does have within itself, and accounted for on its books, all the elements of the full original value which it had at the start, and to which it is entitled.

To illustrate this situation further, one may take, as an example, a railway having extra rolling stock which is only needed at certain seasons of the year when the traffic amounts to a maximum. It must be admitted that such rolling stock is inherently a proper part of the property, but if the owners decided they could spare this extra rolling stock and should sell it, and should depend on replacing it promptly by some unusual exertion when it was actually needed in the time of maximum traffic demand, they would be in very much the same position as they are when they rely on their ability to replace the sinking fund for depreciation when it is needed. In other words, a part of the necessary property "used and useful for the public" is not present and is not accounted for; and, on a valuation or sale, or even for rate-making purposes, they would have to produce this property, or suffer the diminution necessary to the value of the property by reason of its absence. To state it in another way: a utility company plant is not as valuable to the public without a reserve fund for replacement as it is with such a fund intact and on hand and promptly available.

Absent Depreciation Funds as Affecting Rate-Making.—When one seeks to determine the fair return, through proper rates, on the property of a public utility, it is necessary, in order to protect the public,

to determine carefully the value of the property on which such fair return will be allowed; and if it is found that the owners of the property think it a reasonable and proper policy at all times to take out of the property some part thereof, which is ultimately and finally necessary for its maintenance and continuation, and use that part in other ways, perhaps for private gain, one is under the necessity of denying that they can logically earn rates on the part that is removed and separated from the property to which it belongs.

If a sinking fund for depreciation is on hand and attached to the property, then, by reason of the fact that it itself is not a productive fund to the owners, they should certainly earn rates on it. If, however, they assume that they can take the necessary risks and earn more money on such funds elsewhere, and withdraw such funds from the plant and property, then they must of necessity face the contingency that they cannot be allowed by the public to earn rates on those moneys, on the one hand, and use them for private and outside gain, on the other.

It may be objected that it is a hardship on owners of public utilities, who are able and willing to replace in the property such portion of the renewal fund at any moment that occasion might demand, that they should be made to place in trust such moneys and be denied the privilege of their use for outside enterprise; but it will be easily seen that to accord them this privilege does not ordinarily work out with fairness to the public. A public utility owner, under such circumstances, is tempted to let his plant run down, and withdraw from the property the funds necessary for its renewal from time to time, and use such funds for his other gainful purposes. He may then sometime sell his plant (less reserve fund), and the new owner is obliged to furnish what he has not previously withdrawn. The new owner will naturally demand that rates be made such as would net him a fair return on the undepreciated property under these circumstances. Obviously, here, an injustice would be done, primarily to the public and to the new purchaser as well. The whole procedure thus amounts to the withdrawal by the original purchaser of an unearned dividend, and this is rendered possible by the fact that most utility properties have appreciation as well as depreciation, so that, practically, it is the natural accretion that is withdrawn in this manner. The only practicable way to prevent public utility owners from adopting this procedure is to require that the reserve funds for

renewal shall be kept intact as part of the plant and the property; or that the owners be properly fined, so to speak, by the amount of the funds removed or withdrawn.

Outside Investment of Depreciation Funds.—A great deal has been said in defence of outside investment of depreciation funds. Obviously, reserve monies can earn a larger rate of interest in many other ways than in guaranteed reserve funds. A guaranteed depreciation fund, especially for long periods, will not be undertaken by bankers or trust companies at more than a 3½ or 4% rate, and for unusually long periods, say 100 years, the rate must be still lower, say 2½ to 3 per cent.

It has been customary with appraisers to recognize this fact in computing sinking funds on a sliding scale, in accordance with the length of life of the structure to be provided for.

Now, these monies that would be thus set aside in sinking funds are susceptible ordinarily of being invested at much higher rates for general purposes, and where a given result at the end of a given time does not have to be guaranteed, for instance, such monies may be invested as new capital in new construction and extensions in the very plant itself to great advantage, commanding there often as high a return as 7 or 8%, with, of course, the general risk incident to the enterprise.

It is believed by most administrators that this is an entirely proper way to manage such funds, but it is obviously subject to the reasonable conclusion that the owner cannot have a return on it as a "sinking fund" for depreciation, even if he carefully accounts it as such, and at the same time have a return on it as a betterment to the plant. This would be an obvious injustice to the public.

It is also obvious that another drawback to this procedure lies in the fact that the owner must be willing and ready to finance the replacements and depreciation fund, whenever the occasion therefor arises, and make good for such needs as arise in the non-existing depreciation fund. His temptation will easily be to defer and delay, under these circumstances, perhaps to the great disadvantage of the plant and the public, and although he usually has ample notice of future requirements in such funds, it often occurs that the reserve funds are suddenly necessary for accident or emergency.

If this practice of outside investment of reserve funds is to be ultimately allowed by the utility commissions, the owner's problem becomes an economic one, and is only solved by a close analysis as to whether a depreciation fund, earning return and guaranteed by low rates of interest, is more profitable to the owner than the use of the monies elsewhere at higher rates of interest, and the resulting loss of return in connection with the plant. It would appear on the face of the problem that, if rates are to include a return on the sinking funds set aside for depreciation, as well as on the depreciated property, it would be more profitable for the owner to maintain the full fund constantly in actual existence, and it is believed that, when this question has been fully understood and discussed, this procedure will be required by the utility commissions.

Present Practice.—The practice, thus far, where reproduction cost is adopted, has been to deduct a theoretical depreciation fund (where one is not actually in hand) from the reproduction cost of the property new, before proceeding to determine the fair return. The question of the correctness of this procedure was not raised until quite recently, and, like a great many other questions connected with the conduct of public utilities, it will probably have to be reasoned out on lines of fairness and justice to both the public utility owner and the public. The rate commissions have not yet discussed this question, nor has it been effectively presented to them, but it is believed that, when discussed, it will be along lines which are here indicated.

Should a Sinking Fund Earn Full Return?—In pursuing this study, the question will be raised as to the propriety of allowing the same full return on sinking funds, when properly set aside in trust as a part of the plant, as is allowed on the plant value itself.

It may be argued, on the one hand, that such funds are free from the hazards of the enterprise proper, that if properly safeguarded or placed in trust, as they might be, they are especially safe and secure, and are free from risk; therefore, they should be allowed only ordinary going rates of interest for secured investments as their part of the fair return on the entire property, and, of course, entirely outside of their own necessary interest accretions.

On the other hand, it may be contended that such funds, generally, are not put in trust, even where set aside, and therefore are subject to call for emergencies in renewals, and partake, in that way, in all the

hazards and contingencies of the plant and property itself, and should accordingly partake of the full returns which the plant and property itself should have.

Inasmuch as the entire question of returns on depreciation funds has never yet been discussed publicly, the attitude of any public utility commission cannot be known in advance, and we will have to await with patience the conclusions of the older commissions which are giving these matters study.

Actual Use of a Full Depreciation Fund.—It may be observed that it is not theoretically possible to expend properly at any one time the full amount of a correctly computed fund for replacement.

Although actual depreciation requirements are not expected to conform to theoretical depreciation funds, nevertheless the two must be so nearly adjusted that funds for renewal closely follow actual requirements. This can only be accomplished by the use of good judgment and experience, in the first instance, supplemented by periodic revisions, as a secondary aid to proper additions to the fund. It would seem that at intervals of not less than 5 years a careful review of the depreciation account of most utilities should be made, and a revision to comply with new conditions effected; oftener would be better in some special cases.

The fact remains, however, that should funds be correctly set aside for renewal and the life of the various structures of the plant be properly estimated, the funds—either sinking fund or as simple increments—could not be wholly or even largely used at any one time; in fact, ordinarily, only a small portion of such fund will be drawn on at any one time. In a sinking fund, if this were not so, the fund would be seriously impaired, as its integrity, to a certain extent, depends on its accretions of interest. Evidently, the short-lived structures will draw on the fund often, and the long-lived structures but seldom; and, where the bulk of the investment is in long-lived structures, it is evident that at least from one-half to three-fourths of the fund might be untouched ordinarily.

This is an added reason, in long-lived plants, why owners now feel inclined to utilize depreciation funds in outside investments, or in betterment to the plant itself; for the remaining portion of the fund that is frequently called on, being small in amount, is easily replaced, and when the larger amounts begin to be needed the fact is long foreseen.

Depreciation Accounts.—Evidently, the keeping of a depreciation account, without setting aside depreciation funds, amounts to only an expression of opinion by the owner as to the lessened value of his property. Such accounts do not finance the depreciation, but, if properly kept, they may tell how much the owner has withdrawn from the value of the plant, and how much he should return, on demand, to make the plant whole.

Past depreciation may very properly be computed on the sinking-fund basis, because this is the close and accurate method when all or most of the life history is known, but the future is much more uncertain. A conservative estimator cannot figure too closely on it; some allowance must be made for the unknown.

The composite life of many water-works plants, for instance, has been found to be between 60 and 80 years. From 0.6 to 0.8% of the total value of the plant, set aside from year to year as an annual increment and put out at interest as a sinking fund, will usually theoretically amortize the principal at the end of the composite or theoretical life.

It is simpler and more conservative, however, to provide a somewhat larger amount in water-works practice, say 1%, as an approach to a straight-line, or even an interest-bearing fund, which may be called "the reserve". Some of the utility commissions are allowing about this amount to be set aside before computing fair return on water-works properties, and it will ultimately be possible, probably, to count on it. Of course, these rates will vary widely in other forms of public utilities; they are cited here only by way of illustration.

In practice, the various structures having different lengths of life should be grouped, so that those of the same life are aggregated, and a separate depreciation account should be opened for each group. The theoretical sinking-fund amount may be computed for each group and entered each month, so that the sum of all the groups would show the amount due to the plant by the owner at any given time. From time to time adjustments will have to be made to correct for errors in judgment as to future life.

If a simple percentage increment is used, the process will be more simple but less accurate, and hazard and unusual accident may be better and more liberally provided for, but the prophecy as to life will not be closer than it can be in the sinking-fund method.

When renewals are made, the depreciation account should be debited with the outside capital replaced in the plant by the owner, and credited by the cost of the improvement.

Should a sinking fund or a reserve fund actually exist, of course, the replacement expense is directly withdrawn from the same in cash.

Summary.—To conclude, then, it has been argued as follows:

1st.—That if a sinking fund or a reserve fund for depreciation is actually kept in the bank or in trust as part of the property, it should, if properly computed and accurately kept, receive the same rate of fair return as the remainder of the property "used and useful for the public".

2d.—That if these funds are for any reason detached or withdrawn from the property and used by the owner elsewhere, or in private gain, or even as new capital invested in the plant itself, he cannot hope. as a matter of proper protection to the public, to receive return on a reserve fund which is not actually in hand and at the same time use such funds for other personal gain.

3d.—That the past depreciation may be computed accurately on the sinking-fund basis, as all the facts of the past are known, and the sinking fund is the most accurate method of computing depreciation, but for the future, in which the facts are yet unknown, it is perhaps better to use the simpler method of an annual increment which will average the sinking-fund rate.

4th.—That depreciation accounts should be kept by groups having similar life, and revised from time to time as the future is more clearly revealed. That such accounts are only estimates of what the owners owe the property, if the actual funds are not in hand.

5th.—That though it is not now considered improper to use reserve or sinking funds allowed for depreciation for private gain, in so doing the owner will in all probability deprive himself of their return earning power as part of the plant and property.

It may be said, frankly, that all the principles which it is endeavored herein to reason out are not yet generally accepted; indeed, the question of allowing fair return on an actual existing sinking fund for depreciation has not as yet been seriously raised.

It is believed, however, that this question must soon be generally discussed, and it has been the endeavor in this paper to reason out in advance the method by which it is believed it will be logically settled.

DISCUSSION

Mr. Wilgus.

W. J. WILGUS, M. AM. SOC. C. E. (by letter).—The distinction made by Mr. Alvord, that the formal setting aside and labeling of depreciation reserves should entitle the owner of a property to a return on the full cost of its reproduction new, and that the non-observance of this technicality should deprive him of that right, does not appeal to the writer as being just or logical. Surely, if the general balance sheet of a corporation shows an excess of assets over liabilities sufficient to offset the accrued depreciation of the property, there would seem to be no more reason why the return should be based on an impaired investment than that the rent paid by a tenant should be lessened if the owner, who has agreed to keep the premises in repair, elects not to maintain a special fund therefor in the bank.

The majority of railroads have ample profit and loss surplus accounts invested in outside securities or in the property in excess of the amount on which a fair return is earned; and this surplus, for all practical purposes, is a depreciation reserve which insures the ability of the corporation to meet its liability of paying for maintenance out of income. A failure by a corporation properly to meet its liability of maintaining the property out of the rate, is usually the result of over-payment of dividends, and eventually brings its own punishment through the necessity of a reduction or suspension of dividends during the period of rehabilitation. This course is obligatory, as charges for rehabilitation legally can only be made through income.

Whether or not a depreciation fund is actually set aside would seem to be immaterial. What the public is interested in is the proper upkeep of the property without an increase, for that purpose, of the capital burden which it is expected to bear through the rate.

As Mr. Alvord states, "* * * no amount of accounting or theorizing can make good a depreciation fund which is actually not in hand"; but to determine whether or not it is in hand may be done just as effectively through an examination of the balance sheet in conjunction with a valuation of the assets, as in looking for a labeled fund in the bank. To "fine" the owner who elects to protect his liability in some other form than by means of a special fund, seems to be rather a drastic punishment for his selection of one of two alternative treatments of an accounting detail.

Mr. Willoughby.

J. E. WILLOUGHBY, M. AM. SOC. C. E. (by letter).—Few engineers will disagree with Mr. Alvord's suggestion,

"That if a sinking fund * * * for depreciation is actually kept * * * in trust as part of the property, it should * * * receive the same rate of fair return as the remainder of the property 'used and useful for the public.'"

Mr. Willoughby.

The writer, however, believes that this suggested addition to the capital account will find little favor. Depreciation is an operating expense when applied to the accounting of a going public utility, and there is no more reason for capitalizing the depreciation item of operating expenses than for capitalizing any other single item, for instance, the wages paid to enginemen in charge of railway trains.

Furthermore, such funds have not been generally provided as a part of the capital account of our public utilities, and we gain directness by discussing the actual conditions, and the need of providing for depreciation as an operating expense, so that the accounting will always show the value of the public utility for any purpose, rate-making or otherwise.

The accounting of a public utility covers other things than the cost of the plant, and it is the ultimate finding of the accounting (exclusive of the speculative and intangible features) that fixes the value of the utility. The author, however, limits his discussion to that "line of evidence as to value known as the reproduction method, or cost new less depreciation." By thus disregarding all other elements of value that enter into a public utility property, the author has no credit account to offset depreciation, except the cost of reproduction new of the physical property of the utility, and he would limit the rates to be charged by the utility corporation to a reasonable return on the capital necessarily expended for a new plant less the accrued depreciation. This means that the utility corporation must, immediately after reproducing its property new, write off as loss the amount of the accrued depreciation, although that accrued depreciation has actually resulted from service rendered to the public, and, although the only practical way of reproducing the physical property will be to invest first capital to the extent necessary to reproduce the property new.

It is generally accepted that the reproduction of the physical property of a public utility which has been in operation for some years so as to show its exact stages of depreciation would be at a greater cost than to reproduce it with all new materials. Consider the difficulty and expense of providing and assembling into a completed structure the curve-worn rails, the partly decayed ties, the partly fouled ballast, the partly failed bridges, trestles, and culverts, and the argument is completed. Consider then the injustice of the suggestion of the author that the property owner should be penalized because of the impossibility of assembling his plant, except as a structure composed of new elements. What the public pays for is service. If the service is satisfactorily performed, as in the railway after some years of operation, over rails, cross-ties, and ballast, each exhibiting but 55% of their cost new, less salvage, and if to perform that service it has been

Mr. Willoughby.

necessary for the owner to expend in the past 100% for those items, the owner should receive a reasonable return on that 100 per cent. The public's protection lies in scanning the operating expenses to see if the charge for maintaining the property is unduly high, as a result of the failure of the railway company to make provision in the past for the depreciation at the time the depreciation actually occurred, and not to deduct from the capital originally expended the amount of the accrued depreciation. It is only after the operating expenses are deducted that any return can be made to the owners of the stocks and bonds. If the operating expenses are unduly high, on account of failure to make provision for the depreciation as it had accrued in the past, the penalizing of the owner occurs in the denial of deduction from the gross earnings of so much of the alleged operating expenses as is due to the failure of the owner to provide for the depreciation at the time of its accruing. The method of penalizing the owner, as suggested by the author—the deduction of the accrued depreciation from the reproduction cost new—is a confiscation of property to the amount of the accrued depreciation, because the amount of the accrued depreciation was originally and necessarily put into the property before any depreciation could take place, and whatever has taken place is properly to be regarded as an expense of operation. No engineer would deduct from the physical elements of a property the operating losses due to bad financial arrangements, extravagance, or incompetency.

To illustrate further: take a cross-tie costing 80 cents in place, and lasting 8 years. For each year following the placing of the tie, there should be charged out as operating expense the sum of 10 cents (less by such interest as it may earn), as the proportion of the cross-tie consumed in that year's operation, so that when the cross-tie is replaced in the eighth year only 10 cents of its replacement cost would be charged into operating expenses, the other 70 cents being taken from the depreciation fund. The purpose of the depreciation fund is to distribute the operating cost of renewal over the number of years that has been taken to consume the tie. Where a depreciation fund has not been accumulated to offset the actual depreciation, the monetary value of such depreciation will go into the profit and loss account as a debit, but there may be such values in that account—like accumulated reserves—which will entirely absorb the debit. Depreciation, when regarded as an operating expense, can never be used to lessen the reproduction cost of a public utility. It will always affect the value of the utility as a going concern.

Every public utility should make provision in its accounting for depreciation, because such an account is necessary to show the annual operating cost of any plant, and to reflect in the accounting the financial and physical condition and value of the plant. It should cover

"all the losses of value that occur in the property, plants, or parts thereof, from wear and tear, obsolescence, or inadequacy." Mr. Willoughby.

For a public utility that has permitted the depreciation to accumulate without providing for an actual fund to offset the same, then the amount of such accrued depreciation (in such event a debit) will go into the profit and loss account, there to be offset by the credit items of other accounts. Correct accounting does not require the actual presence of cash, except as to the amount of the item of cash on hand.

As to what rates are necessary for a fair return for any public utility, if confiscation is not to result to the property in the legal sense: the suggestion is made that the rates should be sufficient to provide for:

1st.—The payment of taxes and other public dues;

2d.—The payment of all operating expenses, including as a part thereof a charge sufficient to care for the annual depreciation that takes place;

3d.—The payment of a reasonable commercial interest (commercial interest being one that considers and includes the element of risk involved) on the reproduction cost new of the property, including the necessary working capital;

4th.—The payment of a like interest on the intangible values of the property and the unearned increment;

5th.—And, finally, the accumulation of funds sufficient to provide for the non-earning additions demanded by the public. The commercial interest earning additions and betterments can be capitalized and becomes a self-supporting part of the cost of the plant.

Frank C. Boes, Assoc. M. Am. Soc. C. E. (by letter).—After reading Mr. Alvord's paper, the writer tried to picture to himself some of the possible results of putting such a system of rate-fixing into general operation. Mr. Boes.

Consider two adjoining and otherwise similar communities served by two power companies, the one having an old and decrepit plant, but still capable of rendering efficient service, and the other having a new and up-to-date plant. The large users of service, of course, would gravitate toward the community served by the depreciated plant, thereby decreasing property values and rents in the community which they left, and making it impossible for the new plant to compete with the old one. The continual see-sawing of rates, due to periodic repairs and renewals, would make it impossible for service rates to reach a standard level, thus making it impossible for a consumer to foretell what his next year's bill for service would amount to, and would also cause much litigation and confusion.

To fix rates purely on a basis of physical value, would remove incentives for good management and economy, and put a premium on

Mr. Boes. wastefulness, expensive plant, and unnecessary repairs and renewals. Over-expenditure on plant, for the sake of keeping the rate up, is more to be feared than a reasonable amount of decrepitude, for, after all, the service rendered is what is paid for.

If such a system were in vogue now, those who live on Staten Island would be paid a substantial sum for riding on trolley cars which have long outlived their years of usefulness and are run by a plant which is in the condition of the "one-hoss-chaise." They are consoling themselves by the thought that they pay, not toward earnings on a dilapidated outfit, but for the service rendered—a 5-mile ride. A company owning a modern plant might give worse service.

Mr. Alvord states that the average life of the component parts should give the composite life of the plant. This must be an oversight on his part, for, assume a plant to consist of two parts, A, costing \$10 000 and having a life of 5 years, and B, costing \$5 000 and having a life of 20 years. Surely the composite life of the plant is less than 12½ years, for its major part is renewed twice in that time.

In fixing rates by such a method, one must also consider that the depreciation curve is not a straight line. It is permissible to establish a depreciation fund by the straight-line formula, for that is merely a convenient method by which the owner may distribute his renewal costs evenly. The amount of the depreciation fund by no means represents the actual physical depreciation, and to base the deduction of rates on an amount which is taken arbitrarily, and is greater than the actual depreciation, would be unjust, particularly if the service were still unimpaired. When a laborer reaches old age, his earning power is greatly reduced, but is he worth less at thirty-five than at twenty? One cannot expect to deduct \$3 000 from a \$10 000 plant, in fixing rates, and still have a \$10 000 service maintained. If the returns are not based on the entire investment continuously, the investment will not have renewed itself at the end of the plant's term of life, and the owner will not have the plant that he had to begin with. Depreciation is merely another form of capital, and must be kept continuously earning returns on its entire amount.

It seems unjust to expect to pay less for service in the fifth year, the service being exactly the same, merely because some parts of the plant will require replacement at a date then nearer than in the first year. Does one pay less for milk given by old cows or for apples grown on old trees?

Mr. Alvord intimates that the reduction in rates should be considered in the nature of a fine, but surely it would be unjust to have a fine imposed, if, when the time for renewals actually arrived, the owner produced the funds and put the plant in shape. What then? Would the amount of the fine which he had paid be returned to him? A man

cannot be fined because it is thought that he is going to commit a crime. Mr. Boes.

An argument sometimes advanced, and by the Courts, is that one should not make one valuation for rate-making purposes, and another valuation for the purpose of sale or purchase; that is, if valuation is deducted in making a sale of a plant, it should be deducted in making the rates of service. Consider some private business: If an owner wishes to sell his factory, he must certainly make a deduction for depreciation, but the price of his merchandise is not affected in any way by the age of his factory or by the condition of his plant. Neither do his customers inquire as to whether or not he has an adequate renewal fund actually in hand. That responsibility rests with himself alone.

Though there is a difference between fixing the prices in a private business, and fixing the rates of a public utility which enjoys monopolistic privileges granted by the public, the fundamental principles remain the same in each case.

The approaching repairs and renewals are an obligation and a cause of worry to the owner, and to him alone. He has to pay for them, and the consumer is in no way concerned with it, as long as the service remains the same. The consumer is not concerned with how near the day of collapse the plant may be, as long as the collapse never occurs. The consumer is concerned only with impairment of service. When that occurs, then a fine may reasonably be imposed.

In fixing rates, one need only be sure that the property is in satisfactory working order and that proper service is being given. The owners can then be left to worry about the depreciation for themselves.

Allen Hazen, M. Am. Soc. C. E. (by letter).—Depreciation has to be taken into account in both the operation and valuation of public service properties. It has to be taken into account because it is one of the fundamental facts that cannot be ignored. Mr. Hazen.

The writer is familiar with depreciation, especially in water-works properties, and what he has to say relates primarily to them.

Allowances for depreciation have actually been made in nearly all properties of this kind. This is not the less true because the amount allowed has not been determined in a rational manner, and frequently has not even been called depreciation. In most public works the first cost has been paid by money raised by the sale of bonds, and a sinking fund has been established, and annual contributions made to it, intended to be sufficient to provide for the payment of the bonds when they became due, thus, in reality, providing for depreciation, even though the word "depreciation" is not used.

Another way of providing for depreciation which has been common, has been to pay for new construction out of earnings. This also is

Mr. Hazen. an efficient means of allowing for depreciation, although the methods and amounts to make it efficient have seldom been carefully thought out. In general, with publicly owned properties, the allowances that have actually been made for depreciation in these ways have been above the truth. This has been fortunate, as the excess has represented only a slight addition to rates which have not been burdensome, and the resulting strong financial condition of the works has tended to good service, worth more than its cost to the takers.

In the relatively small number of cases where no allowance, or an inadequate allowance, for depreciation has been made, there results sooner or later a cramped financial condition tending to bad service, and equally unfortunate for the works and for the takers.

The measure of depreciation for any period is the decrease in the fair value of the property under discussion, or any part thereof. In valuation proceedings, where the cost of reproduction is being considered, the decrease in the value of the property, as compared with the value of corresponding new property, is the measure of depreciation. The fair value of the property is to be considered always as the fair value between a willing seller and a willing buyer. To take a concrete case, it may be considered how much less a willing buyer would probably pay and a willing seller take because the pipes in the streets had been in use for a time, and had suffered the depreciation naturally incident to use. Obviously, some deduction would be made. The carrying capacity of the pipes is less than that of new pipes, owing to rusting and tuberculation; there is more leakage from the joints in old pipes than in new ones, representing loss of water and profits. It may be presumed that the pipes in an existing system laid from time to time are perhaps less efficiently arranged as to sizes and locations than would be the case in such a pipe system as the buyer would lay for himself. These would probably be the most important points to be considered as between the buyer and the seller in deciding what deduction was to be made. In addition, any peculiar conditions of any kind affecting the value of the property would be duly considered.

The average age of pipes in any system in a growing city will ordinarily be from one-third to one-half of the age of the system, and seldom more than 15 or 20 years. The bare fact that an average of 15 years has elapsed in a possible ultimate life of from 75 to 150 years, which is assumed by the author, would have but little influence between a willing buyer and a willing seller.

The case is somewhat different with boilers, which may be assumed to have a life of 20 years, or some other definite period, and will then have to be replaced. The boiler represents the exceptional case in water-works affairs, and not the ordinary one.

The writer believes that there is no such thing as a period that can be called the life of cast-iron pipe. Different pieces of pipe may have useful lives ranging from a few hours to periods so long that they extend beyond our experience. Any statistics that could be collected would contain records of pipes that had been discarded for one reason or another, and would contain, at best, only estimates of the remaining life of the very important part of all the pipe laid that is still in service. The estimates of the remaining life of this pipe cannot be considered to have much value, and averages based on them are certainly no better. Even if the average life of pipe could be ascertained in some statistical way, it would be of little service. What is really wanted is the average percentage payment on the value of all the pipe to be made each year to make good the depreciation lost in it, and this percentage is undoubtedly quite different from the depreciation computed by the sinking-fund method from the average life of pipe, if it could be ascertained. Mr. Hazen.

The writer believes that in the case of cast-iron pipe the gradual reduction in carrying capacity, the increase in leakage from the joints, and the fact that a system now designed might better serve its purpose than an old system, are the dominating factors, and that the elapsed period of life is of secondary importance.

It may be suggested that in using the sinking-fund method it is possible to assume a life for pipe to give any rate of depreciation that is desired, and as the assumed life is pretty sure to be beyond the experience of the ordinary water-works system, it cannot easily be shown to be fallacious. Thus, with a 4% sinking fund, the depreciations corresponding to several lines are as follows:

30-year life,	1.78%	annual depreciation
40- " "	1.05%	" "
60- " "	0.42%	" "
80- " "	0.18%	" "
100- " "	0.08%	" "

Of course, the percentages given refer to the structures only when new. By the sinking-fund rule they would steadily increase with increasing age.

The ease of selecting a life to give any desired rate of depreciation is obvious. Considering the great difficulty, or perhaps rather the impossibility of securing statistics to show the true average life of cast-iron pipe, it may be questioned whether, as a matter of fact, the process has not been reversed, perhaps unconsciously, by estimators, and a life selected which would give a rate of depreciation which has seemed to them reasonable, as judged by other standards. If this is really the case, then the use of the sinking-fund method for computing depreciation may be regarded as a kind of mental ex-

Mr. Hazen.

ercise tending to familiarize the estimator with the workings of compound interest during long periods, and thereby to aid in keeping him from reaching unreasonable conclusions.

The sinking fund has this to recommend it: that it corresponds with practical experience to the extent that depreciation in the early life of a structure goes on slowly, and that as the years go on the rate is increased. There is a tendency to substitute a rule of calculation for judgment, and, as far as the basis of the rule can be well established, this is desirable; and, if a fixed rule is to be used, probably the sinking-fund method of computation is the safest that has been thus far proposed, but it must not be supposed that the real basis of the sinking-fund method is the actual determined life of the structures. It is, instead, the life which practical appraisers find will give rates of depreciation that are in accordance with their experience.

The writer believes that the most logical way to estimate depreciation in the appraisal of a plant is to consider each item on its merits, to determine and take into account its physical condition, its usefulness in the service and the state of the art, and the possibility of substituting other and more efficient structures to perform the service rendered by it. The excellence of the design must be especially considered, and depreciation may be several times greater on a clumsily designed structure than on a well-designed one, although the durabilities and lives may be the same. In a similar manner, a well-built structure suffers less depreciation than one not so well built.

Improvement in methods of pumping, filtering, laying pipes, or performing any part of the service, all have their bearing on the depreciation of similar parts of an old plant. The rule must always be to find what deduction would fairly be made because of all the conditions that exist between a willing seller and a willing buyer fully informed as to those conditions.

In estimating the amount of depreciation to be written off each year, the writer believes that one of the best kinds of information is to be obtained from the financial histories of old plants in connection with appraisals. The question may then be taken up as to whether an annual depreciation of 1% on the cost of reproduction of the property from year to year, made during the whole life of the plant, would have served to have brought the book value to an amount reasonably near the estimated cost of reproduction at the time of the appraisal, and if not, to find as nearly as may be what percentage thus allowed annually would have sufficed to do this. Data of this kind are hard to get, but the engineer having to do with appraisals occasionally has opportunities to secure them. Valuable as these data are, they must be used with caution, as the depreciation of the future is not necessarily measured by that of the past.

The method of allowing for depreciation in the annual operating account is an important matter. The writer recalls the case of trustees who had undertaken the management of an important water-supply system, and who were confronted on the one hand with the difficulty of raising money for urgently needed extensions, and on the other of being required by law to take up a business entirely foreign to that for which they were appointed, namely, that of setting aside a certain proportion of the income from the operation of the property and investing it in securities to be held against the redemption of bonds when they became due. The situation was a trying one, and one that ought not to have existed. Mr. Hazen.

The sinking fund was invented to protect the bondholders who might think that otherwise the city would not be able to raise so large a sum of money on the maturity of the bonds. That is the only possible justification for its existence. If we grant the solvency of the city or company, and assume that they will be able to refund the bonds when they become due, it is better in every way that the allowance for depreciation, or the sum that would otherwise be put into a sinking fund, should be invested in the plant as far as there is need of it.

The author does not know of any sinking fund that has been established for the especial purpose of meeting depreciation. Such a fund as he suggests would be one to which annual depreciation allowances would be paid, and from which money would be taken to replace worn out or discarded parts, and the amount of such a fund, he thinks, would not often exceed 10 or 15% of the value of the plant. The writer feels that the question of whether such a fund is necessary or desirable must be left to those who operate the plant. If it is desirable, he suggests that it might be more appropriately called "working capital", as its function would be quite different from that ordinarily covered by the term, "sinking fund".

It is the writer's feeling that, so far as such a fund of working capital is reasonably necessary and desirable for the proper and economical administration of the plant, the amount of such fund might properly be taken into account in fixing the rates that might be charged without exceeding those that would yield a fair return on the property; but he sees no reason for taking into account otherwise any part of the value of the original plant that may have been lost by depreciation in making such rates. It is clearly the privilege and duty of public service corporations to charge rates that will permit marking off a proper allowance for depreciation. When this allowance has been once determined and marked off, there would seem to be no equitable reason why the capital thus marked off should be further considered in connection with the question of fair rates.

Mr. Vensano.

TABLE 1.—CASE 1.

			Paid by Public.		Paid by Bank.		Available for Replacements and Reserve.		Available for Dividends.	
			Rate.	Amt.	Rate.	Amt.	Rate.	Amt.	Rate.	Amt.
Original investment	$100.00	Allowed as profits	5%	$5.00						$5.00
		" for depreciation reserve	0.83%	0.83				$0.83		
Sinking fund in bank	33.40	Interest			5%	$1.67		1.67		
Investment in betterments, through later security issues	33.40	Allowed as profits	5%	1.67						1.67
		" for depreciation reserve	0.83%	0.28				0.28		
Total paid by public				$7.78						
" " " bank						$1.67				
		Total for depreciation reserve						$2.78		
Total investment	$133.40	Total available for dividends							5%	$6.67

TABLE 2.—CASE 2.

			Paid by Public.		Paid by Bank.		Available for Replacements and Reserve.		Allowable for Dividends.	
			Rate.	Amt.	Rate.	Amt.	Rate.	Amt.	Rate.	Amt.
Original investment	$100.00	Allowed as profits	5%	$5.00						$5.00
		Allowed as depreciation increment	0.83%	0.83				$0.83		
Depreciation reserve reinvested as betterments	33.40	Allowed as profits on depreciation reserve	5%	1.67				1.67		
		Allowed as depreciation increment	0.83%	0.28				0.28		
Total paid by public				$7.78						
Total investment	$100.00	Available for dividends							5%	$5.00
		Total to depreciation reserve						$2.78		

NOTE.—The total, $2.78, to depreciation reserve, represents $2.50, the constant increment on original investment (after 40 years life) and $0.28, a depreciation increment on depreciation reserve (Case 2), or investment in betterments (Case 1). This is proportional to depreciation increment originally provided on original investment.

out, the returns on a bank sinking fund will be less than the returns from similar funds reinvested in the property itself, and therefore the public would ordinarily actually be benefited by such reinvestment, in that the depreciation increment, which must be included in the rates paid, could be reduced. Mr. Vensano.

This should show clearly that a reinvested sinking fund is certainly not a matter of mere bookkeeping.

It may be argued that the fund is not as safe when thus reinvested, but the assumption must be made that, under public service rate regulation, the rates will be adjusted upward when necessary as well as downward, so that a constant and safe return will be obtained from public utility investments, and, if this is done, the reinvestment of money in the property will be the safest possible method of handling such funds.

If public utility investments are not made safe under Government regulation, Government regulation will certainly fail as such, and either destroy the utilities or necessitate Government ownership; because, with uncertain interest returns, held at a low rate, no capital will be obtainable for investment in such utility.

It may also be argued that, up to the time where dividend returns on the sinking fund have reached the state of balance and become sufficient to cover the annual depreciation replacement, a fund reinvested in the property is not in such shape as to afford ready use as cash.

It is to be noted, however, as Mr. Alvord says, that large replacements can generally be foreseen for a number of years, and a new issue of securities can then be provided to cover such replacement; the betterments constructed under the original sinking fund can be then transferred to capital account under the new securities issued, and the depreciation account credited with the replacement expenditures.

A similar reasoning will hold for any outside investment, provided such investment is safe, and provided the returns from such fund are turned back into it and the whole held as a part of the utility, and, as an additional advisable qualification, that such investment be in liquid form. That is, it might be an investment in bonds of other companies which have a ready sale, and may therefore be considered as liquid. Certainly, no one could object to an investment in Government bonds as being any less safe or less real as a sinking fund than such fund held in bank or in trust.

The writer would also like to add that, in addition to believing that any properly kept sinking fund is a good reason for not deducting depreciation in valuations for rate-making purposes, he also believes that, even in some cases where such fund does not exist, it may be proper to omit the deduction of depreciation. Perhaps it would be

Mr. Vensano. better to say that the writer believes that depreciation in general should not be deducted, except in cases where it is clearly shown that the property has been mishandled by the payment of excessive dividends, or mismanaged in some other way, whereby the accumulating of a sinking fund has been prevented, or when such fund has been mismanaged.

Take, for instance, the case of a company which has gone into business recently in proportion to the average life of its properties and equipment. Assume that such company has been in business 10 years, with an average property life of 30 years. In many cases the property might not have paid during its early life, and with the rates at present in force has just arrived at the point where it is able to set aside money for a replacement fund and to pay dividends. Assume that the earnings are sufficiently large to supply a depreciation increment which in the remaining 20 years of life will give the total fund required, which fund, theoretically, should have been started 10 years previously with a smaller depreciation increment. Would it be fair, then, to such company, which has received no return on its investment, and has been unable, because earnings were too small, to provide depreciation increment, to cut its rates now to a basis fixed by a depreciated valuation. Such property without regulation could no doubt be considered a good and well-handled investment, as business investments go, and still would be placed among the class of failures if its rates were calculated on depreciated valuation, as just shown. In any event, if rates in this case are to be based on such depreciated valuation, it would seem only equitable to make them sufficiently high to afford a depreciation increment large enough to return the entire investment in the remaining life of the plant.

It is admitted, of course, that, in using valuations from which depreciation has been deducted, rates can still be fixed logically and equitably by a proper handling of such depreciated valuation and by allowing earning to cover a depreciation increment based on the remainder of the life of the property, as shown by Mr. Grunsky in the paper to which Mr. Alvord has referred.

The whole question as to whether or not depreciation should be deducted from physical valuations for rate-making purposes would seem to the writer largely a matter as to whether or not the public utility had been handled correctly. It is his opinion that Mr. Grunsky's theory is entirely correct, and, as theory, could be applied to any rationally handled property. It would seem that no theory of any kind can be made to apply to a property which has not been handled reasonably and honestly by its management, and at the same time be made to apply equally to a well-managed property.

Where a property has been earning large returns, and has disbursed them as fast as earned, and over and above a reasonable divi-

dend rate; and where, at the same time, no sinking fund has been provided, it is clear that such company has shown itself ignorant of depreciation results as a whole, and therefore must suffer the penalty. Mr. Vensano.

This, however, would not seem to disprove the theory as such, and as argued by Mr. Grunsky, that, in general, depreciation should not be deducted in physical valuations for rate-making purposes.

LEONARD METCALF, M. AM. SOC. C. E. (by letter).—Mr. Alvord's interesting paper illustrates anew by what different mental paths different men have reached similar conclusions. Mr. Metcalf.

As the writer's views on depreciation are consonant with those set forth by the Special Committee of this Society on the Valuation of Public Utilities in its report which will be in print before this discussion, the writer does not purpose to do more than call attention to certain points in the application of the equal-annual-payment method of figuring depreciation, as compared with the ordinary sinking-fund method referred to by Mr. Alvord. The fundamental difference in the two methods consists in the fact that under the equal-annual-payment method, depreciation is treated as an annual amortization or repayment of capital (renewals *per contra* being treated as new construction), and the necessary accretion figured on the sinking-fund basis is assumed to be paid to the owner with the annual uniform depreciation allowance, figured on the sinking-fund basis; whereas under the ordinary sinking-fund method of treatment the necessary annual contribution is made to a sinking fund which must be kept intact until the amortization of the item of property under consideration, in order that the fund may earn its own accretion. Under the former method the depreciation allowance is actually deducted from the value of the property, and the fair return is predicated on its depreciated value. Under the latter method, although the fair return is predicated nominally on the depreciated value of the property, the sinking fund must be allowed to earn its accretions, and, therefore, in effect, the fair return, until the sinking fund on any item of property is complete, must be based on the full original value, as otherwise the owner will be denied a portion of his legitimate return. An examination of the table in the Committee's report showing the annual depreciation allowances on an item of property having a life of 20 years, will make this clear.

Under the equal-annual-payment method of treatment, the annual depreciation allowance is increased each year over that of the previous year by the amount of interest or accretion on the payment of the last year, increasing in an item of property having a 20-year life, for instance, progressively from \$3.02 per annum on the first year to \$7.64 on the last year, if figured on a 5% rate, the sum of the total payments amounting to 100% in the 20-year life. This is in effect equivalent

Mr. Metcalf. to figuring the depreciation by the sinking-fund method on the original value of the property throughout the period of life of this item, as would be done by the sinking-fund method, it being assumed, however, under the latter, that the sinking fund itself shall earn the accretion.

Why, then, it may be asked, is the equal-annual-payment method to be preferred to the ordinary sinking-fund method? The answer is to be found in the fact that the accounting is perhaps somewhat simplified, and, most important of all, that the combined payments of depreciation and fair return are, with due consideration of the increasing cost of repairs during the life of the various elements of the property, made as nearly uniform as practicable, or, at all events, are made much more uniform than by the sinking-fund method as ordinarily applied.

In its practical application, it has seemed to the writer likely to prove advantageous to establish the approximate allowances to be made for depreciation at intervals of 5 years, more or less, and to convert the estimated allowances or rates of allowance into percentages of gross revenue, rather than percentages of reproduction cost, purely on grounds of expediency and convenience in application. Under such a method it might be found, for instance, that 10% more or less per annum of the gross revenue should be charged off to depreciation or amortization of property—renewals, as previously stated, being charged to new construction. Thus the depreciation allowance would grow annually with the increase in revenue, which is, of course, accompanied with increase in investment and value in property.

To attempt to apply the depreciation allowance as a percentage of reproduction cost involves keeping most careful track of the various elements of property or groups of property having lives of different estimated lengths, adding to and subtracting from these groups new plant built and old plant abandoned—a very laborious and difficult accounting problem. Under the other method of application, book accounts can be kept of the deduction on account of depreciation and memoranda of the property which has actually gone out of service. At the stated periods, then, the more difficult task of comparing the assumptions with the facts developed in the 5-year interval can be accomplished, and any necessary changes in rate allowed can be effected.

Under the equal-annual-payment method and commission control, it does not matter seriously if the allowance for depreciation is somewhat larger than necessary, provided it is not seriously burdensome in the rate, for the reason that the rate-payer immediately benefits in the reduced revenue required, resulting from the amortization of a portion of the property. The owner, on the other hand, does not suffer, as he is repaid an equivalent in cash of the property thus amortized.

Mr. Metcalf.

Although, theoretically, if no new construction were added to the property, the depreciated value of the property would gradually decrease, as also the fair return, practically, no such condition is likely to arise, inasmuch as public service corporation properties are operated perpetually, and with the growth which is enjoyed by the majority of American cities the renewals and the new construction required will more than offset the depreciation allowance, so that the actual depreciated value of the property will practically be likely to increase rather than decrease, as will, therefore, the amount of the outstanding securities of the corporation. If, for short intervals of time, this should not be the case, then the repayment of capital through depreciation account could be carried in the bank or otherwise invested temporarily until the need does arise, or it can be used in actually reducing the mortgage debt by a retirement of a portion of the bonds, which bonds could be re-issued at a later date when the funds are required.

In the sinking-fund method of applying depreciation, the easiest practical method of accounting the sinking fund is probably the investment of the depreciation account in the bonds of the company, if the funds cannot be absorbed in new construction, and the keeping alive of these bonds, with the re-investment of the coupon interest on them in depreciation account.

Mr. Alvord's comment, that if depreciation allowances are not used by the owners of public service corporations in renewals, new construction, or amortization of debt, but are used instead for outside investment, the owners must make recognition of such deduction so far as the public is concerned, is sound; and if these depreciation funds be re-invested in renewals or new construction, the writer is of the opinion that a fair rate of return should be allowed on them as allowed on the remainder of the property, and not simply bank interest rates, for the reason that in such investment in the property, hazard is assumed which must be compensated, and it is further to be borne in mind that there is always some loss in interest in the delay in prompt investment of the sinking-fund surpluses which may accumulate from time to time.

In the case of the equal-annual-payment method, it becomes a matter of indifference to the public how the depreciation funds are utilized by the owners of the property, inasmuch as the value of the property on which the fair rate of return is predicated is decreased annually by the amount of this depreciation allowance, and it seems fair and safe under operating conditions to figure the accretions of annual depreciation allowance to be paid to the owner on the basis of rates at which money could be obtained by the corporation with ample security.

Mr. Jackson.

William B. Jackson, M. Am. Soc. C. E. (by letter).—It is not possible for any one who is truly interested in this subject to read Mr. Alvord's thoughtful discussion without receiving valuable stimulus.

In the case of well-established public utility properties, it does not seem reasonable to consider that a sum of money may with propriety be laid aside in a trust fund of which the amount is determined by the amount which the reproduction cost of the physical property new exceeds the physical value depreciated, in accordance with the rules laid down by Mr. Alvord. There may be possible cases where special conditions, such as hazards of the business, might make necessary the laying aside of a sum of such amount, or even more, but the depreciation element of the property, as defined in this paper—and with which definition the writer heartily agrees—can never justify such a trust fund. As must be readily appreciated by any one who has given this matter careful study, only a relatively small part of such an amount is ever required on account of depreciation in usual public utility properties, and, such being the case, how can any one justly permit of a sum of money amounting to from 10 to 15% of the value of such a property to lie for all time in a trust fund. That is exactly what would be done if a public service company permitted a sum equal to the aforesaid difference to lie in a trust fund, and did not differentiate between the part that might be necessary for use in replacement and that which by the nature of things could never be thus used.

In the matter as to whether, in determining the fair value of a public utility property for rate-making purposes, the reproduction value new or the reproduction value depreciated is more important in arriving at a fair value on which the rate of return shall be made, consider for a moment a public utility property from the time of beginning business. At the beginning the company has a right to earn its current operating expenses, a reasonable sum to cover depreciation expenses, a reasonable sum for reserves, and a reasonable return on a fair valuation of its property. If the company is unable to do this during the early years of its existence, it certainly in equity has a right either to add the deficits to its investment on which it may earn a reasonable return, or to have them made up in some other way. When the time arrives when the company can begin to make money, there is no practical reason to require that it shall build up a larger reserve fund than the exigencies of the business require; therefore, it is not logical to assume that the owners have taken out a part of the value of the plant when they have not built up a depreciation fund to an amount which is entirely unnecessary for the most economical and reliable operation of the property. It is equally illogical to assume that any part of the earnings of the property should be considered as constituting such an unnecessarily large fund.

The writer believes that undue importance is placed on depreciation of plant *per se* without due consideration being given to the question whether the property has been kept in such condition as to give substantially as good and economical service as a new property and is fully safeguarded by its reserves from financial breakdown. Mr. Jackson.

In this the writer does not wish to imply any lack of appreciation of the vital necessity for a vigorous carrying out of a safe depreciation programme, and his position in these matters is fully attested in his paper on this subject early in 1910 before the Western Society of Engineers, and in his discussion written in 1911 for the American Academy of Political and Social Science. There is undoubtedly no more unavoidable expense to be met by any public utility company than depreciation expense, that is, the expense for renewals of outused plant.

F. Lavis, M. Am. Soc. C. E.—It has always seemed to the speaker to be much easier to visualize depreciation and place a value on it than to estimate appreciation properly. He is inclined to agree with the author, that it can be demonstrated, at least from a purely theoretical point of view, that when depreciation is not offset by an actual sinking fund, its value should be deducted in order to obtain the present value of physical property. When, however, valuations of railroads are made for the purposes of rate-regulation or rate-making, it seems as if there might be good reasons for not making the deduction. Mr. Lavis.

The discussion which follows is confined to the effect of this theory on the valuation of the properties of railroads, as the speaker's experience has been principally with this section of the industries of the country coming under the general heading of public service or public utilities companies, and though it is probable that the general principles may hold good in any case, there may be some inherent differences in connection with the properties of some of the other sections, such as lighting, water supply, etc., with which he is not so familiar, which when applied to them might modify the conclusions.

It also seems best to confine any discussion of this subject to cases where the depreciation is normal, that is to say, in the case of railroads, where the plant, roadbed, equipment, etc., is maintained in such condition as to give the public reasonably full and efficient service, which with the present close inspection and regulation by the National and State authorities, should be true of most railroads. Abnormal depreciation, in any event, requires special treatment, as a reduction of rates, of course, would not tend to build up a property which is badly run down.

The theory that depreciation should not be deducted when valuations are made for the purpose of establishing rates has been advanced

Mr. Lavis. by C. E. Grunsky and W. J. Wilgus, Members, Am. Soc. C. E., in papers recently presented to the Society, and by H. P. Gillette, M. Am. Soc. C. E., in a brief recently submitted to the Public Service Commission of the State of Washington. Mr. Alvord also points out other differences of opinion in regard to this matter, and intimates quite truly that a proper understanding of it is not easy.

If a sinking fund is maintained to cover properly such depreciation as there may be, there can hardly be a difference of opinion that rates should be based on the full value; to this the author agrees. If, however, a sinking fund is not maintained, he claims that the value is not in the property, and, therefore, the public should not be asked to provide interest on such full value. He points out also that, in the United States, sinking funds are not usually maintained. In reference to this it may be noted that a sinking fund of 10% on the capitalized value of our railroads would amount to about $1 500 000 000—much more capital than we can afford to have lying idle.

In order to understand the argument of those who claim that depreciation should be deducted, in cases where a sinking fund is not maintained, take a concrete example: for instance, a new railroad, the value of which, for convenience, will be assumed to be $100, and a fair return on the investment as 7 per cent. Assume, also, that the rates provide for the establishment of a sinking fund.

At the end of 10 years the road is rendering full and adequate service, though parts of its structure and equipment show signs of wear. The roadbed, etc., in the meantime has consolidated and its value has appreciated. Assume, however, for the purpose of the argument, that after allowing for the appreciation, etc., the net value of the depreciation is 10 per cent. The actual value of the property would then be $90 and the sinking fund would be $10.

Under these conditions it will be admitted that the rates should still be maintained to provide the 7% on the amount of the full original investment, inasmuch as the sinking fund produces nothing.

During the period under consideration the annual revenue and expense account of the railroad might be approximately as follows: (Round numbers are taken for the sake of clearness.)

Operating cost	$12
Sinking Fund	1
Interest	7
Total	$20

Now, suppose that during this period the dollar, instead of being added yearly to the sinking fund, is paid out in dividends, which have been 8% instead of 7%, so that at the end of the 10 years the $10, instead of forming the sinking fund, have gone back into the pockets

of the owners, then, it is claimed, the public should not be asked to pay rates on the full $100. If the case in actual practice were as simple as this, there would be little need of further argument. Mr. Lavis.

The practice in the United States, however, has not been to establish sinking funds, which produce nothing, but often to put the money back into the property for betterments.

In the case of a new railroad, such as the speaker is considering, as a temporary measure, some of the slopes in the cuts are not fully taken out or not trimmed up until after it is put in operation; the line is not completely ballasted; settlement of embankments has to be taken care of, and so on. Much of this work is done during this first period with the 1% which would have gone to form the sinking fund. Later, the surplus earnings go to provide for reduction of gradients, improvements of alignment, elimination of level crossings, etc.

In Europe and in most other countries of the world, except in North America, all such work is usually charged to additional capital expenditures, and it is largely for this reason that the capitalized value per mile of railroad in Europe is so much greater than in the United States, the capitalized value of English railroads being almost twice as great. There is some idea that railroads in Europe, and particularly in England, have cost more because they are so much better; this, however, is not so, at least not nearly to the extent of the very much greater capitalization.

In regard to betterments and improvements paid for out of earnings, the argument, of course, is that in a properly made valuation of an existing property, the appreciation due to money spent for betterments and improvements will be duly allowed for, and that when it is, depreciation due to actual wear and tear should be deducted, in order to get the true fair value of the property in use.

If appreciation of the thousand and one improvements in the property of any well-established railroad can be properly allowed for, well and good, but, to the speaker, this seems at least somewhat difficult, whereas there is never any difficulty in allowing sufficient depreciation, so that, after all, the question resolves itself into the ability to make a true valuation of the physical property.

There is no doubt that a fair and just valuation, equitable alike to the public and to the owners, can be made, but that it can be mathematically exact is practically impossible, and because there is so much more probability of over-estimating depreciation and under-estimating appreciation, one may very well hestitate before deducting the former from a valuation to be used for the purpose of fixing or regulating rates.

In most of the valuations which have been made public, the so-called present value has been estimated to be from about 10 to 15% less than the estimated cost of reproduction new. In some cases ap-

Mr. Lavis. preciation due to the consolidation of the roadbed, etc., has been allowed, but apparently the net result is to show a present lesser value, and, according to figures generally accepted so far, the normal depreciation of a well-managed and properly maintained railroad is assumed to be about 10 per cent.

It seems to the speaker that this amount might well be, and probably generally is, offset by what might be called "not easily apparent appreciation." Any railway man of experience can easily imagine work of a physical character done on a well-established road, 20, 30, or 50 years old, which is not easily apparent, the worth of which, however, still exists.

In an article* describing the construction of a railroad recently completed near Kansas City, the Chief Engineer makes the statement that "Owing to the newness of the road, the train schedules are not yet as fast as they will be later."

In another article,† describing some recent new work on an improvement of the Nashville, Chattanooga and St. Louis Railway, occurs the following:

"On the new roadbed a heavy bed of cinders is used for ballasting, as this is more easily surfaced during the inevitable settlement of the fills, and forms eventually a good sub-ballast between the roadbed and the permanent stone ballast."

These, of course, are only specific instances of what all experienced railroad engineers know, but it seems as if the importance of this phase of the valuation question is not always fully realized, largely because many of the values thus introduced into a railroad property are not easily apparent.

There is no possible doubt that any of our modern trunk-line railroads are better transportation machines as they stand to-day, with rails and ties partly worn, with other parts of the equipment showing some wear, than they would be if we were able to replace every part in a brand-new condition. Theoretically, by taking specific instances of parts of the railroad machine, such as rails, ties, or rolling stock, one can easily visualize and value the depreciation, but it is known that the well-established railroad, as a whole, is a better and more efficient machine to-day than it was the day it was built.

Knowing this, and there can be no doubt of it, there is something wrong somewhere with a theory which gives a lessened value to a machine which is better than new, and it seems to the speaker that the trouble principally is, not the fact that depreciation is calculated and allowed for, but that appreciation is not appreciated.

In estimating the value, whether for rate-making or for any other purpose, of the purely physical elements of the property of a rail-

* *Engineering News*, October 9th, 1913.

† *Engineering News*, October 23d, 1913.

road, depreciation may be estimated and deducted, but one should also be sure that full value is given to the appreciation, to the development costs, and to the other elements which go to make up the efficient transportation machine. Mr. Lavis.

Mr. Humphreys.

ALEXANDER C. HUMPHREYS, M. AM. SOC. C. E.—The reading of this paper should help to emphasize a responsibility which to-day rests heavily on the engineers of the United States; namely, to educate the public and their representatives in legislatures, commissions, and Courts, many of them necessarily ignorant on the questions involved in valuations and rate-making. Here we, the engineers of the country, are charged with an especial responsibility because of our specialized training. First we should, among ourselves, do our utmost to form right judgments on these momentous questions. It is with great regret, therefore, that the speaker finds himself almost completely out of accord with the views expressed in this paper.

We are to-day passing through a period of trial for our form of government. Therefore the speaker feels particularly that, with regard to all public questions, we must frankly express our honest opinions. This is the only way left to meet effectively the mass of bad advice which is being fed to the public.

In the first lines of the paper it is stated: "that space will not be taken here to review fundamentals". The author then proceeds to discuss the very fundamentals of the subject of depreciation, and to support views which, in the speaker's opinion, are fundamentally wrong.

As the speaker understands the paper, the author is attempting to confine himself practically to "the relation of depreciation to cost new", which, he says, is "vital", and that only one line of evidence (reproduction cost new) is to be considered. The speaker is not prepared to agree that "other lines of evidence are equally important before arriving at value".

Mr. Alvord then refers to some writers who have and have not agreed with him, and continues:

"Here, therefore, there seems to be a very serious disagreement, among authors who have written recently on this subject, as to whether or not depreciation should be deducted from the cost new of a property for finding value for rate-making purposes where the reproduction method is adopted as a basis of value. It seems to be desirable, therefore, to present a further study of this question."

Certainly, this paper makes it more than ever desirable that there should be further study of this question. Although the speaker is overwhelmed at the present time with duties accepted, he feels that he would be delinquent if he did not contribute his little share in this much needed discussion.

Mr. Humphreys.

Mr. Alvord says:

"The sinking-fund method seems to be an accurate way to compute depreciation where all or most of the life history is known, as it consists largely in finding past depreciation."

First, the speaker objects to the use of the term "accurate" in connection with estimates on depreciation to accrue. Again, he objects to the intimation that it can be assumed as a working basis that "all or most of the life history is known." This life history in large part must depend on the books of account and other records, which frequently are incomplete, and, even when complete, have been influenced by many changes in the system of accounting.

Again, in many cases, the existing company is the consolidation of many smaller companies, the books of which are often not available, and if available are found to be not self-explanatory. It would appear as if it should not be necessary to say to men of experience anything more, or even as much as the speaker has said, on this point.

The author then says:

"The sinking-fund method of computing or allowing for depreciation consists in determining the proper useful life of the structure or machine under consideration."

Here he appears to be confusing estimated accruing and accrued depreciation with actual depreciation; but let that be as it may, who shall say authoritatively what is the "proper useful life" of any part of a plant. There are those who assume that the life of parts of plant can be found by reference to tables prepared by so-called experts; but surely this absurdity cannot be supported by men who have had experience as constructors and operators. To determine how much should be allowed for annual expenses or cost, we may well estimate the life of each part of a plant, knowing that these figures may prove to be far from the fact; and hence we are justified in making a liberal estimate for this purpose.

Here, however, we must never forget the underlying reason for this estimate and the journal entries based thereon, namely, to spread the cost of final renewals as uniformly as possible over the periods benefited, so that we shall not deceive ourselves as to losses and consequently as to profits. Especially should we be careful not to over-estimate our profits, and hence we should be sure not to under-estimate the cost of final renewals.

The speaker wishes that the term "depreciation" had never found favor. Much of the present confusion of thought on this subject would be avoided if only the term "final renewals" were used.

Here at once it is seen that such an estimate should not form the basis for determining present depreciation. This should be determined as closely as possible by the exercise of engineering, managerial, and

accountancy ability, based on academic and practical considerations, chiefly the latter. Mr. Humphreys.

Here we have the all-important difference between estimated accruing depreciation as an element of cost, and actual depreciation in establishing the present value of an asset; this last by actual expert examination, having in mind physical decay, obsolescence, and inadequacy.

Again, Mr. Alvord says:

"In case a portion of the life has passed away, the present value of such structure is assumed to be, in a general way, the present cost of replacement new, less the accrued amount of the depreciation fund to date."

The speaker has just shown why the amount in the sinking fund may properly be quite different from the actual depreciation. It is based on an estimate made in advance, and on many assumptions. In making an appraisal we have before us the present facts as to physical condition, obsolescence, and inadequacy. In the first case we have foresight, and in the latter we have the advantage of backsight.

To determine the actual cost, would one go to an engineer's estimate of the cost of some large construction, or to the record of the money actually expended, which would cover all omissions, contingencies, overhead charges, etc.? If the first course were taken, the speaker fears that the business world would be more than ever critical of engineers' estimates, which have so often misled investors because of the omissions referred to.

Supposing the existence of an approximately correct determination of the actual depreciation to date, the speaker contends most emphatically that, although perhaps the plant is thus reduced in value, no deduction therefore should be made in a rate-making case.

In meeting this proposition, the question is asked frequently: "Would you pay as much for an old plant as for a new plant?" and the questioner thinks that the question settles the matter.

Not so. As a purchaser, if there were any real depreciation, the speaker would claim a reduction. Why? Because he has to assume the liability for the final renewal of the plant. In a rate-making case, however, that liability still rests on the owner, and therefore the plant must be considered as maintained or to be maintained by the owner.

Mr. Alvord, strangely enough, admits that "no deductions from cost new or reproduction cost, where that method [actual establishment of a sinking fund under trust conditions] is being used, would be necessary". Then he says:

"Where a proper depreciation fund is not set aside actually, but is only estimated and written off in the accounts, there is no such condition, and the matter of writing it off by the owner in his books amounts

Mr. Humphreys. to only an opinion of his as to what such depreciation fund should be, but does not at all produce such a fund, or make it available; in consequence of which, an appraiser, using the cost new or reproduction method as one of the means of arriving at the value of the property, and finding no depreciation fund actually in hand or in trust, is obliged to deduct it from the property."

Is the appraiser "obliged" to deduct because of an opinion thus expressed?

Mr. Alvord shows that the owner is alive to the fact that depreciation is to be cared for, and that he refuses to inflate his profits by ignoring that fact. By his act he shows that he is liable for the cost of renewal, and that this item of cost must be obtained from income. A buyer would trade on this admission to get a lower price, but, as before shown, even then it by no means follows that the seller would have to deduct the full amount thus written off. That is not a question of the fair determination of values and the fair fixing of rates, but of bargaining.

Mr. Alvord then proceeds:

"In other words, no amount of accounting or theorizing can make good a depreciation fund which is actually not in hand. A clear perception of this principle will save much confusion of thought."

Here Mr. Alvord mistakes his assumption for an established fact. The speaker fears that that is a weakness throughout the paper.

The Standard Dictionary defines "Principle", in part, as follows:

"1. A source or cause from which a thing proceeds; a power that acts continuously or uniformly; a permanent or fundamental cause that naturally or necessarily produces certain results. * * * 2. That which is inherent in anything, determining its nature; essential character; essence."

Here, then, it is found that, notwithstanding his disclaimer, as before referred to, Mr. Alvord is discussing fundamentals.

The speaker presumes that much confusion of thought might be saved if all were prepared to adopt blindly Mr. Alvord's dicta and then refuse to confuse their minds by further independent thinking.

Now, to come down to facts: Assets are widely variant in character. Cash is an excellent asset, if the money is honest money. Particularly is it valuable in emergencies because it is liquid. Usually, it is a promise to pay, on the part of the government, of a bank, or, if not in actual cash, it might be a promise to pay by the trustee of a fund; but government or bank promises to pay are not always made good. There has been no little repudiation, even in this enlightened country. Or, government notes may be greatly depreciated. We have also had that experience in the United States, a condition corrected only by the courageous act of a great President now passed to his reward. Still

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more, the trustee of a fund may default—not an unknown occurrence. Now, if the owner is solvent and agrees to make good the loss due to depreciation, that is, to renew the plant, and this assumed liability is shown on the books, there must be a corresponding asset, or else money not earned has been paid out in dividends. The entries referred to, however, have been made to guard against such a ruinous course. Naturally, the speaker is not referring to fraudulent bookkeeping or fraudulent management, but is following Mr. Alvord in assuming normal conditions and management.

This asset, if not in cash retained, will in all probability appear in additional plant paid for from earnings, and may be regarded as borrowed from the depreciation reserve. The fact is that where the depreciation reserve is treated in this way it has to be borne in mind that all parts of plant which appear in the appraisal inventory as having been paid for from the depreciation reserve fall in one of two classes:

1.—Parts which have been installed as renewals or replacements, and are, therefore, in place of parts represented in the original investment, and hence to be included as such;

2.—Parts of plant which have been installed as extensions or betterments, and have been paid for with money borrowed from the depreciation reserve.

These parts, therefore, should be included in the inventory and appraisal, because they are not duplications of investment, but represent additional investment.

In the case of an interest-bearing depreciation reserve, the balance to the reserve should be credited each year with interest at the rate agreed on. Then the money thus borrowed has to be returned when required for final renewals, and must be repaid from capital. In the meantime the company has been able to defer the day for final financing. These extensions and betterments can be considered as capital investments, the amounts borrowed from depreciation reserve standing as a liability against the owners. In reference to this matter, the point may be made again that, whether or not there is a depreciation reserve, the liability for renewals rests against the owners. The setting up of the reserve is, as before shown, to spread this item of loss as uniformly as possible over the periods benefited. The loss due to final renewals will occur in any case, though not exactly in the amount estimated.

Mr. Alvord then attempts to show that the deduction for depreciation would not lead to a progressive reduction in the rates. This discussion he carries on under the headings, "The Attempt to Simplify the Question", "Perpetual Life", "Utilities Commonly Require Grow-

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ing Plants", "Earning Power of the Depreciated Plants", "Natural Accretions", and "Sinking Funds an Inherent Part of the Plant".

The speaker agrees with some of the statements he makes under these headings; but disagrees absolutely with his conclusions. On page 796 the author states:

"It is clear, therefore, that, as far as valuation for the purpose of sale is concerned, it is right and proper that, in the absence of actual sinking funds or any sufficient reserve fund, they should be deducted from reproduction cost or 'cost new', when that method is used to ascertain value. Does not this also hold good in the case of valuation for rate-making?"

It is presumed that the author looks for a reply in the affirmative to this question. The speaker unhesitatingly replies in the negative; and this, it is thought, is where Mr. Alvord and his many adherents go astray.

The case of purchase and sale has nothing to do with the question, and the author has supplied, presumably unconsciously, many arguments to support this position. What he has to say under "Perpetual Life," can serve as an example.

What the public is concerned in is the quality of the service rendered. That is the test. It is for the commissions to see that the service is adequate in quantity and quality and that the investment and its return are not exorbitant.

In purchase and sale, it is a case of bargain. It may be a case of turning a fixed asset into a liquid asset. There may be many reasons for the owner's willingness to sell well below value; but in any case he should be willing to deduct fairly for accrued actual depreciation, because the buyer has to assume the liability therefor. If the seller can turn over a sinking fund, which in the opinion of the buyer is a sufficient offset to depreciation, then his liability therefor is covered. All this provided this is the best bargain he can make; for bargain it is.

Under the heading, "The Practical Treatment of Depreciation Accounts", Mr. Alvord takes, as an example of the point he is trying to demonstrate, the case of "a railway having extra rolling stock which is only needed at certain seasons of the year when the traffic amounts to a maximum". Then he shows that the property would not be as valuable if this extra rolling stock were sold, relying on the chance of buying it back when actually needed. Again, the test is one of service. If this rolling stock had been sold before the appraisal were made, and not replaced, naturally the decrease in value would be reflected in the appraisal. If all the property had been sold, it would have a still greater effect on the appraisal, because no appraisal would be necessary.

To wind up this part of the discussion Mr. Alvord says: Mr. Humphreys.

"To state it in another way: a utility company plant is not as valuable to the public without a reserve fund for replacement as it is with such a fund intact and on hand and promptly available."

This is not stating it in another way, but is an entirely different proposition. The reserve fund is a matter of finance. The cars might not be purchasable for immediate delivery, but this by no means shows that money for replacements could not be obtained simply because there was not cash enough on hand. Such a case is almost unbelievable, provided the owner enjoyed the ordinary good credit of public service corporations. Again, it is a question of service, and the public is not interested in the financing of the replacements provided the amounts thus expended do not add to the capital burden.

Under the head of "Outside Investment of Depreciation Funds", Mr. Alvord objects to investing the depreciation reserve in betterments which might give 7 or 8%, instead of the guaranteed 3½ to 4 per cent. He refers to "the general risk incident to the enterprise". As all the capital invested is subject to this risk, there is no good in splitting hairs about this particular part of the investment; but Mr. Alvord thinks that it works an injustice to the public if the owner gets a return on these betterments. If the return has been allowed for in the amount annually to be set aside, it does not. The returns are in lieu of the interest otherwise allowed by the banker who holds the depreciation fund, and these returns are required to bring the reserve up to the required amount. As to the extra rate of earning, there can be no injury to the public if the property prospers because of this or any other saving. Also, the speaker imagines that Mr. Alvord has failed to appreciate the fact that, as a rule, extensions thus made do not pay a high return at the start and are frequently made in advance of a paying demand for the service.

The speaker, as a manager, has had to face this problem in not less than fifty cities in the United States.

Under the head of "Present Practice" Mr. Alvord says:

"The rate commissions have not yet discussed this question [that of deducting a 'theoretical depreciation fund'], nor has it been effectively presented to them, but it is believed that, when discussed, it will be along lines which are here indicated."

Believed by whom? The speaker quite disagrees as to the correctness of the statement. He can hardly think of any question which has been more discussed in the cases of which he knows. He also ventures to believe that the questions involved have been "effectively presented". The statements have not always been understood, for those to whom the statements and arguments were addressed were in some cases, through lack of training and lack of experience in the several lines

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of construction and administration, incapable of understanding the matter presented.

Under the head of "Should a Sinking Fund Earn Full Return", Mr. Alvord says: "We will have to await with patience the conclusions of the older commissions which are giving this matter study". Again, the speaker completely disagrees with him. It is not for us to "await with patience", or with impatience, the conclusions of the older commissions, or the younger commissions, on any point. It is for us to go ahead with open minds to determine what we believe is fair and honest to the public and public utilities, and to do our best to educate the public and the commissions. If they are right and we are wrong, it is for us to acknowledge the fact; it is for them to do the same. It by no means follows that we are to accept as correct the rulings of the commissions simply because we have to govern ourselves thereby until these decisions are reversed. Even the Supreme Court of the United States is not infallible.

The speaker also objects to the reference to the "older commissions", as if they of necessity were most likely to give just decisions. That does not follow. There may be an old commission with a new personnel, and "the last state may be worse than the first".

Under the head of "Depreciation Accounts", Mr. Alvord indicates that in his opinion the cost of depreciation should be estimated liberally. He says, "Some allowance must be made for the unknown"; also, "From time to time adjustments will have to be made to correct for errors in judgment as to future life". This is quite true; but the unknown has not happened when the rate-making appraisal is made, and if no "adjustments * * * to correct for errors in judgment" have been made, how about deducting from cost new the full amount thus accumulated in the reserve?

In the speaker's experience the so-called experts testifying for the people deduct the depreciation according to an assumed "average life", and they do not worry about the "unknown" or "errors in judgment". They are so used to dealing both in the unknown and errors in judgment that they have long since ceased to worry.

On page 803 Mr. Alvord says:

"When renewals are made, the depreciation account should be debited with the outside capital replaced in the plant by the owner, and credited by the cost of the improvement."

Is this one transaction which is referred to? Is it a replacement? If so, is it an improvement? Of course, a replacement may include in part the element of improvement, but this hardly seems to be what is intended. To what account is "the cost of the improvement" to be credited?

The speaker does not find this a self-explanatory direction for a journal entry, and would suggest that it be rewritten.

Now, as to the several items of the summary: Mr. Humphreys.

1.—This is incomplete.

2.—The speaker disagrees with this, absolutely.

3.—"All the facts of the past" are not known. The sinking fund is not the most accurate method of computing depreciation. The depreciation has to be estimated before the sinking fund can be computed.

4.—This is quite correct, and furnishes an answer and contradiction to much that is in the paper.

5.—The reference to "private gain" is not clear. If the speaker understands the statement, he considers it an unfair proposition.

In his concluding words, the author says: "all the principles which it is endeavored herein to reason out are not yet generally accepted".

The speaker is glad to be able to agree with Mr. Alvord here; but must go farther and express the hope that most of these "principles" never will be "generally accepted". They cannot be accepted without working confiscation in some degree.

In conclusion the speaker begs to refer to a paper recently written by him which discusses at length most of the questions raised by Mr. Alvord. This paper is entitled "Depreciation, Estimated and Actual." It was presented in June, 1913, before the Institution of Gas Engineers of Great Britain, and has been reprinted in many of the technical journals of the United States, and also in pamphlet form.

In this paper the speaker supports the proposition that no deduction should be made for depreciation if the property has been adequately maintained, and particularly that no deduction should be made on account of aging of the plant. He draws the line sharply between estimated accruing depreciation and actual present depreciation, the latter as found by expert examination at the time. He also shows that in not a few cases there is no need for any estimated depreciation because the current final renewals make a fairly uniform annual charge, and this applies to properties as a whole, and in many more cases to parts of properties, so that those parts can be eliminated from the estimate of accruing depreciation.

Henry Floy, M. Am. Soc. C. E.—Without discussing in detail the points made by Mr. Alvord, some of which are well taken, the paper, as a whole, is wrong in its conclusion and unfair both to the utilities and the public. Mr. Floy.

The author bases his paper on two assumptions—which he does not attempt to prove—both of which are false:

First.—That both newly originated and long established utilities must be treated alike.

Second.—That a depreciation reserve fund is a necessity.

Mr. Floy.

Before undertaking an intelligent discussion of this subject, there must be drawn a sharp line of demarcation as to whether the valuation being considered is that relating to an existing and long established corporation, or one which has originated under public service regulation. The same rules may not fairly be applied equally to utility corporations so differently originated.

In a case of property constructed and put into operation before the creation of regulating commissions, the investors, with the consent of municipal, State, or Federal authorities, and the sanction of the public at large, were allowed to do certain things which are no longer permitted under utility regulation. Where a corporation is created and the money of the owners is invested, after notice, under conditions prescribed in advance by regulating bodies, there can be no fair or honest objection to the insistence on accumulation of depreciation funds out of revenue and the use of such funds in any reasonable way, but they cannot necessarily be used as a measure of the value of the property on which rates are to be fixed. With respect to corporations that have been in existence for a considerable period of time without commission regulation, and have been following certain generally accepted principles with the consent, more or less definite, of public authorities, the speaker takes emphatic exception to the proposition, advanced by the author, that value for rate-fixing purposes must be determined by deducting the amount of a computed but not accumulated reserve fund from the cost of reproduction new. The arguments presented in the paper may be reasonable when applied to some newly organized corporations, but they are unfair as applied to most existing corporations. The point to be made clear is that no *ex-post-facto* rulings should be attempted by commissions or countenanced by such organizations as the American Society of Civil Engineers, simply because no reserve fund is in hand. Who shall say that, because utilities had not accumulated reserve funds before public regulation was instituted, therefore, the corporations shall now be deprived of a part of their property. In determining rates where the public and corporations have consented to the existence of a certain thing in the past that is not legally or criminally wrong, and about which there still exists honest differences of opinion, it is not fair or right, in making rulings for the future, to attempt to apply such rulings to the past at the sacrifice of investment or the jeopardy of income. It is on this point—the difference between a corporation already in existence and one originated under new laws—that the mistakes have been made in the rulings of commissioners who, for the most part, inexperienced in utility corporation affairs, have attempted to apply theoretically perfected methods which are largely the result of development and experience. It is the old truth, that the same medicine will make some men well and others ill.

That the so-called, though misnamed, present value is not determined by the cost new less the amount of depreciation or reserve fund is indicated by the fact that a fund accumulating for a period of years, based on the original investment, would probably result in too small deduction from the reproduction cost new in case of investments which have enormously increased in value, and too great deduction in the case of those which have largely decreased in value. Mr. Floy.

In the second instance it may certainly be questioned whether there is any necessity for the accumulation of a reserve fund for the purpose of ultimately returning to the investor full value of his property, except in a case of limited franchises or where the lives of utilities are definitely determined. Although in most cases a moderate reserve fund should be kept in hand for contingencies, it is a fact, as has been argued for some years by the speaker, that there are utilities which do not require the accumulation of any depreciation reserve funds whatsoever. A corporation which has its property widely distributed, of various characters, with single units of relatively small value, may not require any depreciation reserve because the deterioration of property—after the first few years of operation—is taking place at an approximately uniform rate per annum, and, therefore, all depreciation becomes wear and tear and is taken care of as a part of the annual operating expense; as an example the Third Avenue Street Railway Company, of New York City, may be cited. This property consists of nearly 300 miles of surface track, partly overhead and partly underground trolley, with rolling stock to correspond, of all sizes and types, as well as storage battery cars, underground conduits, cables, overhead transmission systems, car barns, sub-stations, generating station, real estate, and other property extending from the lower end of the Island of Manhattan up to and through the Bronx and the County of Westchester, so that the exhaustion of any physical property, as far as can be anticipated, will not result in unduly increasing operating expenses for its replacement, or affecting net income, or in any way jeopardizing service or charges to the public. Despite the orders of the Public Service Commission to do so, the Third Avenue Company accumulates no depreciation fund, and yet is operating most successfully.

Why, then, should Mr. Alvord argue that this company should have on hand a reserve fund equal to 15 or 20% net value of its property, say, from $9 000 000 to $15 000 000, for which the corporation has no use, except to be allowed to earn fair return on this property under his theory.

On page 798, Mr. Alvord says, "a public utility owner, under such circumstances, is tempted to let his plant run down, and withdraw from the property the funds necessary for its renewal from time to time, and use such funds for his other gainful purposes." Except

Mr. Floy. under abnormal conditions, this statement is not in accordance with the facts. Valuations of a large number of utility properties of different kinds, made by a host of independent and separate appraisers, result in showing that the average condition of these properties is between 80 and 90% of reproduction value after deducting the amount of depreciation, as determined by methods suggested by Mr. Alvord, or the straight-line method of calculating depreciation which shows even a smaller present value than Mr. Alvord's method. Consequently, it may be stated, without fear of contradiction, that the condition of the average utility plant is not run down, whether or not reserve funds are maintained. Moreover, utility regulation provides for proper maintenance and service, and this can be enforced and secured even to the exclusion of dividends. Take the case of a corporation which has issued only stock, and has no bonds or mortgage indebtedness outstanding—as is not unusual, particularly in New England—in such a case, the entire interest of the stockholders is a guaranty of good service. Why, then, must the public be further protected by an unwieldy and useless reserve fund, which is of no service to the corporation, except to permit it to earn on the full value of the property?

Mr. Burns. Clinton S. Burns, M. Am. Soc. C. E. (by letter).—The question as to whether or not depreciation should be deducted from the cost new of a property in finding the value for rate-making purposes where the reproduction method is adopted as a basis of value, is one that has often suggested itself to the writer, but not until recently has he undertaken to make a comprehensive analysis of the subject with regard to the equity of the points involved. Considering the case of the most simple proposition, as cited by the author, where the composite life of an entire plant has been estimated, at the end of which the plant would be subject to entire renewal; that is to say, the original investment will have completely disappeared at the end of this estimated life, the following analysis suggests itself:

First, let it be understood that the obligation to renew the property, or any portion thereof, is a liability of the public utility company, and if the company has not the cash in hand with which to meet this liability, it must assume the burden of financing the necessary renewals whenever the necessity may arise, and as far as the public is concerned, it is immaterial whether it chooses to provide in advance for the financing of renewals by setting aside its annual accretions to the depreciation fund, or, whether it chooses to make other uses of these funds and then re-finance the renewals when the necessity arises—provided, however, that it is strong enough to accomplish such re-financing promptly when called on to do so. Furthermore, it is now conceded by all authorities that the obligation to renew the property, or, in other words, to make good the depreciation, is very properly transferred to

the public (the rate-payers). This is expressed in substance by the Supreme Court of the United States in the Knoxville case in the following words: Mr. Burns.

"The Company is not bound to see its property gradually waste without making provision out of the earnings for its replacement. It is entitled to see that from the earnings the value of the property involved is kept unimpaired, so that at the end of any given term of years the original investment remains as it was at the beginning."

This language clearly defines the underlying principles which should govern an analysis of depreciation. Depreciation is an obligation to be met by the rate-payers—the public—at the time when renewals become necessary. Stating the problem in its simplest form, freed from all legal entanglements, and considering only the equity in the case, the following analysis results:

An investor, on entering the public utility business, dedicates his property to the public use, and, in return, the public obligates itself to pay a reasonable rate of return for the use of the property and to restore to the owner the value of any portion of it consumed or wasted in the public service; or, in other words, the public is obligated to make good the depreciation. Suppose, now, that the utility, at the beginning of the enterprise, should accept from the public the promissory notes of all the rate-payers, payable on demand, without interest, the face value of these notes aggregating an amount equal to the reproduction cost of the property, it being stipulated that the payments would be demanded only as required for the renewal of totally depreciated property. If such a procedure were practical, and assuming that these notes were convertible into gold dollars on demand, at full face value, depreciation would thus be provided for; and an appraiser, called on to determine the value of the property for rate purposes, would undoubtedly recognize the injustice of deducting depreciation from the cost of reproduction in any project financed in this manner. Now, the foregoing procedure is exactly what the sinking-fund theory is, except that the investor allows these promissory notes to be paid in small annual payments, just sufficient so that if invested at compound interest, the total sum accumulated at the maturity of the notes (the expectancy of the property life), will equal their face value. The investor, therefore, in accepting depreciation on the sinking-fund basis, does not in fact, so far as its use value is concerned, have any of his available capital returned to him until the end of the life of the structure, and in order that he may receive a fair return on the value of his property, the rates must be based on the reproduction cost of the property, undiminished by depreciation. If the investor chooses to reinvest the cash on hand in the depreciation fund at any time, with the hope of securing greater returns therefor, he must assume the risk and incur the liability of returning the amount withdrawn

Mr. Burns. plus its accumulations at compound interest when necessary for renewal expenditures; for example, if he is collecting depreciation on the sinking-fund basis, where 4% is considered a fair rate of interest at which to compound the fund, and he is able to withdraw the fund and invest it in extensions to the property, or elsewhere, and thus realize 8% thereon, the additional 4% is due him as compensation for the effort, risk, and worry inherent to the investment of capital, and the first 4%, being the proper rate for liquid capital, must be considered as going to restore depreciation.

The writer fails to see why any distinction should be made between the case where the sinking fund is actually kept in the bank or in trust and where it is kept merely as a book entry and has been diverted to other investments. In fact, the public, by the fundamental principles on which the sinking-fund theory is based, compels the utility owner to invest the depreciation fund somewhere, where it will draw 4% compound interest, or some predetermined rate considered proper for liquid capital; and how can it matter to the public whether it is invested in time deposits in a bank, in Government bonds drawing the necessary rate of interest, or in private venture where the owner perhaps hopes to realize 10%—and what matters it whether or not his expectations are realized? The public secures its recompense in being permitted to discharge its depreciation obligation by paying into the sinking fund the smaller annual contributions and imposing on the owner the obligation of reinvesting the fund; and the owner secures his reward for reinvestment if this venture proves successful, and he pays the penalty if it is a losing one.

The author states further "that the owner cannot have a return on it [the sinking fund] as a 'sinking fund' for depreciation, * * *, and at the same time have a return on it as a betterment to the plant. This would be an obvious injustice to the public." At first thought this seems plausible, but, on further consideration, is it not a fact that the contrary is the case, namely, that the public in reality owes this opportunity for reinvestment to the utility owner?

The public, having imposed on the public utility owner the necessity for reinvestment of the fund where it will earn, say, 4% interest, should in equity open the avenue of investment to him when the opportunity presents itself. To illustrate further, take the simplest case imaginable; for instance, a gravity system of water-works, where the entire property consists of \$100 000 worth of wood pipe, having a life of 20 years; the utility owner dedicates this \$100 000 to the public service, and, in return, the rate-payers obligate themselves to pay a reasonable return on the investment, and to preserve the property intact; or, in other words, make good the waste from usage. The rate-payers provide for the depreciation by giving their promissory notes aggregating \$100 000 in face value, payable 20 years after date,

without interest. The utility owner permits these notes to be paid in annual installments on a 4% sinking-fund basis, payable at the beginning of each year, or at the rate of \$3 229 per annum; and it so happens that the betterments, which in this instance are for extensions to the pipe system, require the expenditure of \$3 229 per year. The rate-payers, in consideration for being allowed to care for depreciation by paying the sum of \$3 229 annually, permit, and, in fact, compel, the utility owner to reinvest this sum of \$3 229 annually where it will earn 4% compound interest, and they also require him to invest this or some other \$3 229 annually in pipe lines for extending the service to new territory; but they pay him extra in consideration for these new extensions an amount equal to 3.229% to restore depreciation and, in addition thereto, a reasonable return on these further sums invested annually. Mr. Burns.

Two courses are open to the investor: he may leave the first \$3 229 annually in a trust fund, earning 4%, in some reliable bank, and use \$3 229 annually of his other resources in extensions, or, he may expend the \$3 229 depreciation fund directly for extensions. In the former case he will find his situation in 20 years to be as follows:

He will have \$100 000 in the trust fund with which to replace the original pipe system, and he is also in possession of \$100 000 worth of extensions paid for from his outside resources. In the latter case, he will find at the end of 20 years that he has \$100 000 worth of pipe requiring replacement, with no funds in trust, but he has the \$100 000 in his outside resources which he may as well now spend all at once instead of at the rate of \$3 229 annually, as was done in the former case. He also has the same investment in extensions or betterments as in the former case. Hence, the owner's financial situation is exactly the same at the end of 20 years regardless of which method of finance he chooses to pursue. The public is likewise in identically the same position in either case; for, had the owner left the depreciation account in trust until the \$100 000 had accumulated, at the end of 20 years that sum is due the owner in exchange for his worn-out property, dedicated and consumed in the public service. Hence, the public could have no claim on the sum, nor title to any portion of it. Neither the owner of the utility nor the public being in any wise affected by the owner's withdrawal of the sinking fund for reinvestment or for extensions of the property, it would seem proper that the valuation for rate purposes should be the same in either case, no matter whether the sinking fund is left in a trust fund or used for private venture. If the amount of depreciation to be set aside annually is computed on the sinking-fund theory, it seems, therefore, that no deduction can equitably be made from reproduction cost in arriving at the proper valuation to use as the basis of rates. The rate-payers, therefore, should contribute annually \$3 229 for depreciation and the reasonable

Mr. Burns. rate for interest and profits to the owner, regardless of whether or not the depreciation fund is held in reserve.

The writer, therefore, can see no reason in equity for the conclusion that any injustice to the public results from allowing the owner of the utility to have return on the sinking fund for depreciation at the proper rate of interest for such depreciation accounts, and at the same time have return on it as betterment to the plant, for each betterment to the plant simply means a larger investment or larger plant on which the owner is entitled to the proper reasonable rate of return. Doubtless what the author had in mind was that it would be unjust to allow the owner to reinvest the sinking fund in betterments and receive thereon, say, 8% net return, and, at the same time, receive 4% as sinking-fund accumulation. It seems to the writer, however, that when the owner invests his sinking fund in betterments, or in private enterprise, and thereby receives 8% return, he in fact does not receive 8%, but, on the contrary, receives only 4%, because half of this 8% return does not represent profit but simply goes to restore to him some of his property already consumed by the public in the public service.

If a community constructs a utility under municipal ownership, it is almost universally the custom to pay for such a utility by a bond issue. Now, to determine the basis of rates in such a municipal enterprise, in order that it shall be made exactly self-supporting, with no surplus, the amount of the original bond issue is always taken as the basis on which the community must pay interest, assuming that the sinking fund with which to pay off the bonds is based on an expectancy equal to the life of the property; that is to say, if the bond issue runs for the number of years represented by the life of the property, the community must set aside a sinking fund sufficient to pay off the bonded indebtedness, and must likewise continue to pay interest on the full amount of the bond issue, regardless of the fact that the property itself is continually depreciating in value. In fact, this method of financing the enterprise is absolutely essential if the project is to be made self-sustaining. How, then, can there be any escape from the conclusion that a privately owned utility must receive its return on the same basis, for to receive less will undermine the integrity of the investment, and is, therefore, not a fair return. Of course, if the municipal bond issue is to be paid serially, then the amounts paid off from time to time would be deducted from the capital on which the public must continue to pay interest, but in order to pay these bonds serially, the annual levy or contribution for sinking-fund purposes must be largely in excess of the amount necessary for the application of the sinking-fund theory. It would seem, therefore, that the proper deduction to make from reproduction cost is simply the amount by which the actual depreciation allowance exceeds the theoretical accu-

mulation in a sinking-fund account, computed on the age and life of the property. If depreciation is figured on the sinking-fund basis only, therefore, no deduction should be made from reproduction cost. Mr. Burns.

The Public Utilities Act of California, in effect March 23d, 1913, provides, in Section 49, that:

"The Commission shall have power, after hearing, to require any or all public utilities to carry a proper and adequate depreciation account in accordance with such rules, regulations and forms of account as the Commission may prescribe. The Commission may, from time to time, ascertain and determine and by order fix the proper and adequate rates of depreciation of the several classes of property of each public utility. Each public utility shall conform its depreciation accounts to the rates so ascertained, determined and fixed, and shall set aside the moneys so provided for out of earnings and carry the same in a depreciation fund and expend such fund only for such purposes and under such rules and regulations, both as to original expenditure and subsequent replacement as the Commission may prescribe. The income from the investments of moneys in such fund shall likewise be carried in such fund."

With the administration of the depreciation fund in accordance with this Section of the California Act, it would seem that such fund must be kept separate and set aside, and as far as the owner of the utility is concerned, he has no discretion whatever over its disbursement, and he cannot withdraw such fund for any purpose other than for the replacement of depreciated property. Therefore, depreciation cannot equitably be deducted from the cost of reproduction in California rate cases when that method is used to determine the value of the property.

Section 47 of the Public Utilities Act of Idaho, and also Section 79 of the Public Utilities Act of Missouri, use almost exactly the same language as the California Section already quoted. This indicates the tendency in recent legislation to require public utility companies to set aside sinking-fund accounts, or, in other words, it takes these accounts out of the hands of the public utility owners and leaves them largely under the control of the public through its representatives, the public service commissions. There can be no objection to leaving the collection and disbursement of the sinking fund entirely under public control through proper utility commissions, and, in fact, it is highly desirable that such should be the case, but it is important that the commission should study carefully the conditions surrounding each property in order that the amount of depreciation collected from the rate-payers, and thus set aside, shall be adequate and equitable under all the circumstances.

Contingent depreciation should receive special consideration in each case, its amount depending on the nature of the property. Contingent depreciation is the sum that should be provided from earnings

Mr. Burns. to meet unusual or unexpected replacements which become necessary from time to time in the experience of every public utility, such, for example, as damage from floods, destruction of property by earthquake or ice gorge, failure of water supply, and many similar experiences. Depreciation by obsolescence or changes in the art should also be fully covered by the allowance for contingent depreciation. In fact, contingent depreciation may be defined as the sum that should be provided for insurance against such contingencies as cannot be covered by any class of commercial insurance. This raises an important question: Is the sinking-fund method of computing the depreciation fair to the owner of a utility in a rate-regulation investigation? Evidently not, unless the rates are based on the cost of reproduction undiminished by depreciation, but, under the usual procedure in a rate investigation, it is extremely difficult, if not utterly impossible, to receive consideration for the full reproduction value of a public utility, and would it not be more practical, therefore, to adopt some system of accounting depreciation that will secure equity to both parties when considered in conjunction with a proper rate of return on the remaining expectancy of the property? Perhaps this can be understood more clearly by considering the property as passing through a series of successive ownerships. It cannot be denied that the owner may sell the property at any time and retain the accumulated depreciation fund. This fund is due him, for, if he sells the property at its fair value, computed on the sinking-fund basis, he must retain the depreciation fund in order to recover the full amount of his investment.

Consider again the case in its simplest form, the gravity system of water-works previously cited, and, for the purpose of simplicity, assume that no extensions or betterments are required throughout the 20 years' life of the property, and that no elements of appreciation in value accrue to affect any portion of the works. The first owner should receive from the rate-payers the sum of \$3 229 for depreciation and a reasonable return on his investment of \$100 000. He sells the property at the end of the first year at its value, computed on the sinking-fund basis, namely, \$96 771. The second owner should receive from the rate-payers for depreciation the sum of 3.475% of his investment of \$96 771, or \$3 362.79—this being computed on the basis of the remaining life of the property, 19 years—and, in addition thereto, he should receive a reasonable rate of return on the reduced capital account, \$96 771. Now, if such a rate is equitable to each of the successive owners of the property, it must be equally fair to the public and to the owner throughout the life of the property, if the same ownership continues throughout the entire period. For the purpose of illustration, Table 3 is submitted, computed on the basis of 4% compound interest on the depreciation fund, 8% for interest and profit on the capital investment, and a variable allowance for contingent

depreciation, starting at 2% of the capital account and increasing as the property becomes older, the amount of increase being such as to preserve a constant total contribution from the rate-payers. It is desirable, from the point of public expediency, that rates be not fluctuating, and this may be accomplished by the variable contingent depreciation, based on an amount that shall represent, as nearly as can be predetermined, the facts in each case under consideration. Mr. Burns.

TABLE 3.

Owner.	Capital invested.	Depreciation. Rate. Per cent.	Depreciation. Amount.	Interest and profit. 8%.	Contingent depreciation.	Annual cost to rate-payers.
1st........	$100 000	3.229	$3 229	$8 000	$2 000	$13 229
2d........	96 771	3.475	3 363	7 742	2 124	13 229
3d........	93 408	3.749	3 502	7 473	2 254	13 229
4th........	89 906	4.058	3 648	7 193	2 388	13 229
5th........	86 258	4.406	3 801	6 901	2 527	13 229
6th........	82 457	4.802	3 960	6 597	2 672	13 229
7th........	78 497	5.257	4 127	6 280	2 822	13 229
8th........	74 370	5.783	4 301	5 950	2 978	13 229
9th........	70 069	6.399	4 484	5 606	3 139	13 229
10th........	65 585	7.130	4 676	5 247	3 306	13 229
11th........	60 909	8.009	4 878	4 873	3 478	13 229
12th........	56 031	9.086	5 091	4 482	3 656	13 229
13th........	50 940	10.435	5 316	4 075	3 838	13 229
14th........	45 264	12.174	5 554	3 621	4 054	13 229
15th........	40 070	14.496	5 809	3 206	4 214	13 229
16th........	34 261	17.753	6 082	2 741	4 406	13 229
17th........	28 179	22.643	6 381	2 254	4 594	13 229
18th........	21 798	30.803	6 714	1 744	4 771	13 229
19th........	15 084	47.134	7 110	1 207	4 912	13 229
20th........	7 974	96.154	7 667	638	4 924	13 229

An examination of the column in Table 3 headed "Contingent Depreciation", shows that the variation in contingent depreciation from year to year is not great, under this system of accounting, and represents very closely what may be reasonably expected, increasing gradually with the age of the property. The contingent depreciation is not a cumulative fund, but, on the contrary, represents the amount actually spent from time to time in replacing obsolete or abandoned property and in repairing damage from unexpected or unusual causes. If properly proportioned, such a fund will fluctuate between surplus and deficit, but will maintain a general average near the zero line. The actual figures used in Table 3 are, perhaps, not appropriate for a property of this nature, but are submitted simply to illustrate a principle and without reference to the adequacy, or otherwise, as this can be determined only by a study of all the circumstances surrounding each particular property. The table is based on annual contributions to a sinking fund at the beginning of each year, although many authorities prefer using the tables computed for the end of the year. Rates are usually collected quarterly or monthly and, therefore, as

Mr. Burns. a matter of fact, it would more nearly accord with correct theory to use tables computed for annual contributions in the mid-year, if such tables were at hand. However, this difference is not sufficient to cause material error in any event.

Unfortunately, in actual practice, the problem is never so simple as the gravity water-works system used in this illustration. Public utility enterprises are, from the very necessity of the case, almost continuously extended from year to year, so that the average age of a property of this nature never even remotely approaches that of its oldest parts.

If the theory outlined herein is accepted as a correct method of computing depreciation, its practical application would be somewhat as follows:

The appraiser would determine the composite life of the utility under consideration, which, for illustration, in the case of a water-works system, in some particular case might be found to be 60 years. The average age of the entire property, computed in like manner, might be found, for example, to be 20 years. The rate to be used in determining the amount of ordinary depreciation therefor would be based on an expectancy of 40 years. On a 3% sinking-fund basis, this would be 1.288%—this percentage to be applied to the remaining capital determined by deducting the sinking-fund accumulation of 20 years from the cost of reproduction of the property. The proper additional amount to be allowed for contingent depreciation, of course, should be determined by the particular circumstances in the case. It would be more precise to compute the result separately for each group of items having the same age and estimated life; but such a procedure would, perhaps, be impractical in most cases, owing to its difficulty of application.

Mr. Knox. STUART K. KNOX, M. AM. SOC. C. E. (by letter).—Mr. Alvord's paper, dealing with "the general relation of depreciation and its effects on the fair return of a utility property," will have justified itself if it does no more than precipitate the extended discussion of this interesting and intricate subject, concerning which ideas are at present so confused and conflicting. In this, it appears to the writer, will lie its chief, if not its only, value.

As the author implies, there exists at the present time a wide diversity of opinion among engineers, lawyers, special masters, commissions, and Courts having to do with valuations for rate-making purposes, with regard to the fundamental question of the value on which fair rates should be predicated.

Among those who have sought to sustain the reproduction-cost-new theory may be mentioned the special master,* *re* Columbus Railway

* Robert H. Whitten. "Valuation of Public Service Corporations," p. 368.

and Light Company *vs.* City of Columbus, Ohio, 1906; the Wisconsin Railroad Commission,* *re* City of Whitewater *v.* Whitewater Electric Light Company, decided December 16th, 1910; Massachusetts Joint Board,† Massachusetts Appraisal of the New York, New Haven, and Hartford Railroad, 1911; C. E. Grunsky,‡ M. Am. Soc. C. E., in his paper entitled "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates", to which paper the author refers; and Alexander C. Humphreys, M. Am. Soc. C. E.,§ President of the Stevens Institute of Technology, in his paper entitled "Depreciation: Estimated and Actual", presented before the Institution of Gas Engineers of Great Britain, 1913. Mr. Knox.

The contrary view, to the effect that fair rates should be adjusted to yield a fair return only on depreciated reproduction-cost, has been upheld by the Oklahoma Supreme Court,‖ *re* Pioneer Telephone and Telegraph Company *v.* Westenhaver, decided January 10th, 1911; by the New York Public Service Commission for the First District,¶ *re* Metropolitan Street Railway Reorganization, decided February 27th, 1912; and by the United States Supreme Court.

Mr. Alvord's ideas on the subject are summarized in his first and second "conclusions," as follows:

"1st.—That if a sinking fund or a reserve fund for depreciation is actually kept in the bank or in trust as part of the property, it should, if properly computed and accurately kept, receive the same rate of fair return as the remainder of the property 'used and useful for the public'.

"2d.—That if these funds are for any reason detached or withdrawn from the property and used by the owner elsewhere, or in private gain, or even as new capital invested in the plant itself, he cannot hope, as a matter of proper protection to the public, to receive return on a reserve fund which is not actually in hand and at the same time use such funds for other personal gain."

The writer is not prepared to admit the truth of either of these propositions without material qualification, and in the following discussion will seek to show:

(*a*) That the disposition made of the depreciation reserves has theoretically no bearing whatever on the question of value on which the owner of a utility property is entitled to earn a fair return.

(*b*) That the question of value on which an owner is entitled to earn a fair return depends solely on whether the depreciation reserve provided has been or is to be just "adequate", "excessive" in amount, or "inadequate" for the purpose intended.

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* *Loc. cit.*, p. 366.
† *Loc. cit.*, p. 363.
‡ *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 770.
§ *Transactions*, Institution of Gas Engineers (Great Britain), 1913, p. 348.
‖ Robert H. Whitten, "Valuation of Public Service Corporations," p. 375.
¶ *Loc. cit.*, p. 376.

Mr. Knox. He will endeavor to demonstrate that when the depreciation reserves, which have been and are to be provided, are just "adequate" for the purpose intended, the owner must, if he is to be protected against the impairment of his capital or of its earning power, be permitted to earn, in addition to operating expenses and this "adequate" depreciation reserve, a fair return on reproduction-cost-new.

That when the depreciation reserves, which are to be provided, are "excessive", so as virtually to effect a refund to the owner of a portion of his original investment, together with its full earning power, rates may be equitably adjusted to yield a fair return on a value less than reproduction-cost-new, the measure of the reduction being the total amount of the capital so refunded.

That when the chosen method of providing for future depreciation is such as virtually to effect a progressive refund, free and clear, of a portion of the owner's original investment, a corresponding progressive decrease in the value on which he is allowed to earn a fair return may be equitably made.

That when the depreciation reserves which have been provided are "excessive," so as virtually to have effected a refund of a portion of the owner's original capital, rates may be equitably adjusted to yield a fair return on a value less than reproduction-cost-new only if no change in ownership has taken place.

That when the depreciation reserves which have been provided are "inadequate", so that an actual impairment in capital has taken place, we should, to be consistent, and provided full ownership is still retained by the owner who suffered the loss, permit the adjustment of rates to yield a fair return on a value greater than reproduction-cost-new until this loss has been recouped.

That rates may be equitably adjusted to yield a fair return on depreciated reproduction cost, only if the virtual refund of capital, resulting from "excessive" past or prospective depreciation reserves, just equals the accrued depreciation on the property on the date of the valuation for rate-making purposes.

As a preliminary to the intelligent discussion of the preceding propositions, it is essential first to define exactly what is meant by "adequate", "excessive", and "inadequate" depreciation reserves, and by the expressions "accrued depreciation" and "depreciated reproduction cost".

By "adequate" depreciation reserve is meant the annuity or percentage of reproduction-cost-new which, if set aside in the bank or in trust from the beginning of operation, at the rate of interest, say 4%, on which the fund payments are predicated, will, with all interest accretions, amortize a sum which, at the termination of the useful life of each element of the property, will just suffice to replace it and no more.

By "excessive" depreciation reserve the writer means an annuity greater in amount, and by "inadequate" depreciation reserve an annuity less in amount than the above. Mr. Knox.

By "accrued depreciation" at any period of the life of a property, is meant the amount which an "adequate" depreciation reserve would have amortized had it been set aside each year from the beginning of operation.

By "depreciated reproduction cost", is meant the reproduction-cost-new less the "accrued depreciation" so determined.

By the "useful life" of any element of property, is meant its age plus the remaining life expectancy of the element.

In explanation and justification of the foregoing definitions, it may be well to digress at this point to the extent of acknowledging a full realization of the fact that any estimate of accrued or prospective depreciation is at best a guess. There are, however, intelligent guesses and unintelligent guesses, and there are intelligent ways of guessing and unintelligent ways of guessing. One unintelligent way of guessing the accrued depreciation on a pumping engine, for example, is to sit in the office and compute it, starting with the assumption that the normal life expectancy of a new unit of the type considered is 20 years, and that the age of the particular unit under consideration is, say, 10 years. The unit may have been in continuous service from the date of its installation, or it may have been used largely as a spare. It may have been operated 24 hours per day, or only 1 or 2 hours per day. It may or may not have had intelligent care, and it may or may not be well adapted to the service, well designed, and of modern type. No intelligent guess of accrued or prospective depreciation can be made without a visual field inspection of the element to be appraised.

Whatever be the age of a unit, or structure, the present value of which it is desired to ascertain, the uncertain element affecting this value which may be guessed most accurately by an expert is its remaining life expectancy. In the light of the history and probable future of the plant, of the community which it serves, and of the arts, this, after an examination of the unit or structure, may be gauged as closely as any other probability affecting the future of the property. Then, if the guess proves accurate, the "useful life" of the element becomes its age plus its life expectancy on the date of the appraisal; and an "adequate" allowance to cover annual depreciation is the annuity, which, had it been paid into a fund each year since the installation or construction of the unit or structure, would have amortized a sum sufficient to reproduce it at the end of this "useful life". Under these conditions the "accrued depreciation" on the date of the appraisal may fairly be estimated as the amount which such an annuity would have accumulated between the date of installation and the date of the appraisal, and its fair market value or depreciated reproduction-cost

Mr. Knox. at the date of appraisal may fairly be assumed to be its reproduction-cost-new less the accrued depreciation so found. Fair market value under these circumstances would of course be defined to be the value as established between a seller who was willing but not compelled to sell and a buyer who was willing but not compelled to buy.

In all that follows, it will be assumed that the guesses as to life expectancy and hence of proper or "adequate" depreciation reserves are accurate ones.

To come to the discussion of Mr. Alvord's conclusions, it is undoubtedly true as the author states that "there must be in this, as in all other questions relating to this difficult subject of valuation, a just and equitable relation between the public and the public utility owners, which is founded on rational analysis and common sense." In seeking out this relation much is gained in the way of clarity by confining ourselves at first to the consideration of simple examples. We are thus able to eliminate the many complicating, confusing, and often entirely irrelevant considerations which usually cloud and obscure the vital principles involved, when the question is studied in the light of complex cases such as commonly come before the Courts. Although it is no doubt true, as the author implies, that conclusions deduced from the consideration of simple examples will not always admit of indefinite extension to cover the complex cases met in actual practice, we seem at least to be on safe ground in assuming, as a point of departure, that a conclusion which will not withstand the most searching analysis, when viewed in the light of its application to a simple practical example, cannot be the basic principle or relation which we are seeking. At any rate, let us grant the truth of this proposition for the moment, and observe the result of applying such a test to the conclusions reached by Mr. Alvord, beginning with his first conclusion, "that if a sinking fund or a reserve fund for depreciation is actually kept in the bank or in trust as part of the property, it should, if properly computed and accurately kept, receive the same rate of fair return as the remainder of the property 'used and useful for the public'."

If by a "properly computed and accurately kept" sinking fund, Mr. Alvord means an "adequate" depreciation reserve, as defined in a preceding paragraph of this discussion, the logic of this conclusion appears to be unassailable.

Consider, for example, a water-works plant, consisting solely of a pipe line 10 miles long. The owner of this property, we will say, buys water at wholesale at one end of the pipe line, and sells it at wholesale at the other end. The writer has in mind a private water company the property of which is substantially as described. Assume that the reproduction-cost-new of the pipe is \$1 000 000, its useful life 50 years, and its scrap value nothing. No part of the line will require replacement during its 50-year life, and the pipe may

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be maintained in good serviceable condition throughout the entire period by making such ordinary repairs as may be properly charged to operating expense. Neglecting the fact that the creation of a sinking fund must lag one year behind the commencement of operation, the proper annual reserve to cover depreciation, estimated on a 4% sinking-fund basis, is \$6 550, which amount the owner places in bank each year at 4% compound interest.

In the construction of this pipe line, the owner converted \$1 000 000 of money into physical plant. As the pipe ages, it depreciates in value, and if the guess as to its useful life (50 years) proves correct, the amount of this depreciation is, as nearly as may be, represented at any time by the accumulation in the sinking fund or reserve fund for depreciation. The process may be regarded as the gradual re-conversion of plant value into cash, until at the end of the pipe's useful life the owner has all cash and no pipe line, at which time the money accumulated is again converted into physical plant through the taking over of a new pipe line which the owner has caused to be constructed. At any intermediate period of the life of the pipe, the owner's capital is represented partly by the depreciated value of the line, and partly by the depreciation reserve. The sum of the two may be considered as representing at any time the reproduction-cost-new. A purchaser can at any time afford to pay for the pipe line and depreciation fund together, a sum equal to the pipe's reproduction-cost-new, for by continuing the depreciation reserve in bank, and adding the proper annual payments, as these are earned, he will have available at the termination of the useful life of the property a sum equal to his original investment and to the pipe's reproduction-cost-new. This is equivalent to saying that a purchaser can at any time afford to pay for the pipe line alone a sum equal to its reproduction-cost-new less the accumulations in the sinking fund, or its depreciated reproduction cost.

Now, as all interest accretions, if the annual depreciation reserve is a proper or "adequate" reserve computed on the sinking-fund basis, must be credited to the fund, the owner enjoys no income therefrom, and, therefore, should be allowed to earn a fair return on the depreciated physical value plus the depreciation reserve, or on the reproduction-cost-new. To assert the contrary is to assert that the owner must, in spite of everything he can do, suffer a financial loss through impairment of the earning power of the capital which he has invested.

Let us now examine Mr. Alvord's second conclusion:

"That if these funds [adequate depreciation reserves] are for any reason detached or withdrawn from the property and used by the owner elsewhere, or in private gain, or even as new capital invested in the plant itself, he cannot hope, as a matter of proper protection to the public, to receive return on a reserve fund which is not actually in hand and at the same time use such funds for other personal gain."

Mr. Knox.

Suppose that the owner of the pipe line operates his plant in exactly the same way as described, except that instead of actually placing \$6 550 in bank or in trust each year as part of the property, he invests this amount in other ways, or even places it in a safe deposit vault, or squanders it. Mr. Alvord concludes that under these circumstances rates may be equitably adjusted to yield a fair return only on the depreciated physical value of the pipe instead of on its reproduction-cost-new. This means that unless the annual depreciation reserves are correspondingly increased above the "adequate" amount previously estimated, a sliding scale of rates must be introduced, such that the amount available for interest in the tenth year of operation, after deducting operating expenses and depreciation reserve, will be approximately 92%, in the fortieth year approximately 38%, etc., of its amount in the first year of operation. As the service is now by hypothesis exactly the same as in the case where the depreciation reserve was actually deposited in bank as part of the property, the water consumer will evidently gain in reduced rates by the owner's failure to establish an actual depreciation reserve. It is equally evident that the consumer cannot gain unless at the same time the owner loses an equal amount. Let us pursue the illustration further.

Suppose that the owner, in the tenth year of operation, becomes alarmed at the dwindling profits on his water transportation business, and begins to busy himself in devising a plan for restoring rates to the point where they will yield the same return on his capital as in the first year. He estimates that if he had from the outset maintained a depreciation fund in the bank at 4%, the accumulations in this fund would at the end of the tenth year have amounted to \$78 760.56. He, therefore, sells at par from his private means seventy-nine \$1 000 municipal bonds drawing 4% interest, or he sells, we will say, an apartment house valued at \$78 760.56, which has been yielding him a net return of 4% per annum. He deposits the proceeds, \$78 760.56, in the bank at the beginning of the eleventh year and christens it a depreciation fund. He has not lost a dollar by this transaction. The depreciation fund is his property in the same way that the house was his property. The interest yield is the same. He has simply performed the familiar operation of taking a sum of money out of one pocket and placing it in another, and yet, as a result of this simple shifting of assets, he has placed himself in a position where he may equitably charge his customer, the water consumer, we will say, an additional \$10 per 1 000 000 gal. for exactly the same quantity and quality of water, delivered under exactly the same pressure as before the depreciation fund was created. Could anything be more absurd?

Mr. Alvord speaks of the use of the depreciation reserves, by the owner, "for other personal gain". It is, of course, apparent that when these reserves are proper or "adequate" reserves computed on the sink-

ing-fund basis, it is entirely beyond the power of the owner to use them so as to yield "a private gain", unless he invests them in a way to yield a return greater than that which they would have earned if placed in the bank or in trust at the rate of interest on which the sinking or depreciation fund payments were computed. Mr. Knox.

If correctly computed on the sinking-fund basis, the sum of the annual reserves will, by hypothesis, produce an amount sufficient to provide for future replacements, only if credited with all interest accretions resulting from their conservative investment. The source of these interest accretions is immaterial. The funds are equally unproductive, as far as concerns their power to yield a "private gain" to the owner, whether they be placed in bank at 4% interest, invested in high-grade municipal bonds at 4% interest, or invested in other ways. If the owner is fortunate enough to invest them in a way to yield a return greater than they would have earned if placed in a bank or in trust, his "private gain" is represented, not by the total yield of the funds so invested, but only by the excess of this yield over and above what they would have earned if placed in the bank or in trust. To obtain this excess yield or "private gain", the owner performs services and assumes risks for which he is entitled to compensation represented by the excess yield. He has, in fact, entered the banking business and is as much entitled to a fair profit on this banking business as he is on his water-works business. In his capacity of water-works manager, he deposits with himself, in his capacity of banker, the depreciation reserves on which he pays 4% compound interest. When the time arrives, he must return these sums with accumulated interest, the same as any other banker would be obliged to do. In the meantime he is entitled to invest them conservatively in such a way that they will yield enough more than 4% to compensate him for performing the services and assuming the responsibilities and risks which attend the banking business.

There are bankers and trust companies, many of them, that actually own and operate water companies. Is it to be presumed that these banking institutions, to avoid being "fined," must invest their depreciation reserves with some competitor, perhaps less responsible and competent than themselves? Again, the proposition is absurd.

Theoretically, the consumer has no interest whatever in the depreciation reserves or in the disposition which is made of them. He is interested only in the quality of the service. Service is what he pays for. He pays, not for the quality of service which an element of plant is capable of rendering when new, or at its absolute maximum of efficiency, but for the average service which it is capable of rendering during the period known as its "useful life," when maintained throughout this life in the highest state of efficiency which is physically possible, and replaced at its termination with a new ele-

Mr. Knox. ment. In the absence of certain knowledge to the contrary, it must be presumed that the owners will so maintain and so replace the plant, regardless of whether or not an actual, tangible depreciation fund is maintained in a bank—a competitor's bank. Even when depreciation reserves have been distributed to stockholders, or lost or tied up in unfortunate investments, it must be presumed, in the absence of certain knowledge to the contrary, that the owners will provide funds as required by dividend reductions, assessments against the stock, or in extreme cases by reductions in interest.

As a practical matter, it is granted that in certain cases the owner's failure to invest depreciation reserves conservatively may react on the consumer, and that to this extent he is interested in the disposition which is made of them. It is, for example, a matter of common observation and knowledge that when an owner first becomes involved in financial difficulties there exists the temptation and tendency to maintain dividends and interest at the expense of maintenance and replacements. The prevention of this is one of the functions of the regulating power.

As long as the quality of the service is actually maintained, there can exist no logical reason why the value on which the owner of a property is entitled to earn a fair return should be variable, depending on whether or not a reserve for depreciation is maintained in the bank or in trust as part of the property. To penalize or "fine" an owner for failure to maintain an actual fund by adjusting rates to yield a fair return only on depreciated reproduction-cost, is to presume in advance of the fact that he will not, when required, and at his own cost, make the replacements for which depreciation reserves were provided. It is in effect to punish him in advance for an offense that it is suspected he may commit at some time in the future.

The points just made have been clearly stated by the Wisconsin Railroad Commission *re* City of Whitewater and Whitewater Electric Light Company, decided December 16th, 1910.* The Wisconsin Railroad Commission says:

"As it is a general rule that the reasonable return which a utility is allowed to earn covers the interest and depreciation on the actual investment in the plant, it becomes important to know what the investment in the plant actually is—that is, what is the value of the plant new. The fact that the property of the utility has diminished in value with use, as the inevitable result of depreciation, does not lessen the amount of the investment in the plant. To be sure, it may happen in the case of a given utility that money which should have gone to the establishment of a depreciation fund has been diverted to the stockholders, thereby apparently lessening their investment. If an amount equal to the difference between the value of the plant new, and the value in present condition is thus paid over to stockholders, it would

* Robert H. Whitten, "Valuation of Public Utility Properties," p. 366.

appear, at first sight, that the value of the plant in present condition would be the basis on which interest returns should be allowed. But it must not be forgotten that at the expiration of the life of the plant if the money which should have been used to provide for depreciation has been paid to stockholders in the form of dividends or otherwise, the value of the plant will be nothing. Then, instead of the utility having a depreciation fund on which to draw to replace the plant, the owners will find it necessary to pay the cost of replacement, presumably from the money which they have received from the plant, but which should have been used to provide a depreciation fund. The investment in the plant then must, in general, be taken at the cost of the plant new, since although the investment may apparently be diminished by failure to provide for depreciation, and by the payment of this money to owners or stockholders, in reality the investment is not diminished, because of the necessity of replacing the plant, in the absence of a depreciation fund, from the property of owners or stockholders. Therefore, it appears that the question of valuation, which is of most importance in the case, is that of cost of the plant new, or the actual value of the total investment in the plant." Mr. Knox.

It is not necessary to state that a reduction, below reproduction-cost-new, of the value on which an owner is permitted to earn a fair return, may be logically justified only on the assumption that a virtual refund of a portion of the capital originally invested has taken place prior to the date of the valuation for rate-making purposes. It has been shown that no such refund is effected where an owner, in lieu of maintaining an actual tangible depreciation fund in the bank, invests these reserves in other ways. There are, however, ways in which such a virtual refund may occur, and it may prove interesting to examine one or two of them.

Suppose, for example, that the owner of the pipe line, previously referred to, instead of setting aside each year in addition to operating expenses and interest an "adequate" depreciation reserve of $6 550, sets aside a depreciation reserve, improperly computed on the straight-line basis, of $20 000 per year. Under these circumstances interest accretions are no longer, of necessity, credited to the fund, for the sum of the annual payments themselves, will, without interest, accumulate an amount sufficient to replace the pipe line at the end of its useful life. Whether placed in the bank as an actual tangible depreciation fund, or invested in other ways, the owner may each year draw and add to his gross earnings the income from this fund without in any way defeating the object for which it was created. However it be invested, the entire fund may now be used productively by the owner "in private gain." A portion of his original investment equal in amount to the sum of the annual depreciation reserves has in effect been refunded to the owner by the consumer. It has been refunded free and clear, together with its full earning power, and if the owner is to be prevented from receiving, at the expense of the consumer, a double interest return

Mr. Knox. on the same principal, rates must be adjusted to yield a fair return, not on reproduction-cost-new and not on depreciated reproduction cost, but on reproduction-cost-new less the sum of the "excessive" annual depreciation reserves. This value will be less than the depreciated reproduction cost. At certain periods during the life of the pipe, it will be materially less. At the end of the twenty-fifth year, for example, the "accrued depreciation," or the sum amortized by an "adequate" depreciation reserve of \$6 550 per annum, will be only \$272 803, whereas the sum of the annual depreciation reserves, without accrued interest, will be \$500 000. The depreciated reproduction cost on that date will be \$1 000 000 — \$272 803 = \$727 197, whereas the value on which the owner should be allowed to earn a fair return will be only \$1 000 000 — \$500 000 = \$500 000, or about two-thirds of the depreciated reproduction cost.

If an owner has, during the past, set aside out of earnings an "excessive" annual depreciation reserve, or if he has, what amounts to the same thing, enjoyed excessive profits, it may appear from the preceding discussion that the ends of justice will be served by adjusting rates to yield a fair return on a value less than reproduction-cost-new. The justice of the procedure, when adopted as a retroactive or punitive measure, is, however, only apparent. The ownership of most public utility properties is constantly changing through transfers of stock, and reprisal would in many cases fall, not on the original owners, who alone could be supposed to merit it, but on the present owners who purchased in good faith, at a fair price, and reaped no benefit from past extortions.

Suppose, for example, that at the end of the twenty-fifth year, and just prior to the date of a valuation for rate-making purposes, the pipe line, previously referred to, had been sold by the original owner, A, to the present owner, B. Suppose that A, throughout the period during which he operated the line, had extorted from the public, in addition to a fair return on his capital, improper depreciation reserves estimated on the straight-line basis, which amounts he did not transfer to B with the depreciated property. A has now placed himself beyond the reach of reprisal. He has gone out of the waterworks business, taking his unholy profits with him. He has sold his plant for its market value which, for the purposes of this discussion, may be assumed to be fairly represented by its depreciated reproduction cost at the date of sale (\$727 197). He could not have sold it for more however moderate his early charges for depreciation had been. B has purchased the property in good faith and for a fair price. It might now be argued superficially that as far as B is concerned, the price paid represents his total investment, and that the adjustment of rates to yield a fair return on this purchase price will impose no hardship on him; but this is not the case. The fallacy in the reason-

ing, though perhaps not obvious, lies in the assumption that the money which B has paid to A for the physical property represents his total investment in plant. It does not. A has sold to B a depreciated physical property plus a liability. The liability consists in the obligation which B incurs of replacing the depreciated pipe line at the end of its useful life. To place himself in a position to meet this obligation when it matures, B must, on the date of the purchase, set aside in bank at 4% interest an amount ($272 803) equal to the accrued depreciation on the plant. If he fails to do this, he, nevertheless, automatically, by the assumption of the obligation, ties up an equal amount of his outside investments as far as concerns their power to yield him a "private gain." In any case his total investment is not represented by the sum of money paid to A, but by this amount plus the accrued depreciation. Whatever disposition he makes of the depreciation reserve, if it is only an "adequate" reserve as previously defined, B will inevitably suffer a financial loss unless allowed to earn a fair return on $1 000 000, the reproduction-cost-new. Any reduction in this value will inevitably work a great injustice by causing B to suffer for the sins of his predecessor. Mr. Knox.

B, it is true, will be protected against loss, if at the same time that rates are adjusted to yield a fair return on depreciated reproduction cost, he is allowed to set aside each year from earnings, an "excessive" depreciation reserve sufficient to reproduce the depreciated plant value ($727 197) at the end of its 25 years of remaining life. It may indeed be laid down as a general rule that where past depreciation reserves with interest accretions have been adequate to take care of the accrued depreciation, and future reserves are made sufficient in themselves to amortize sums which will provide against all subsequent deterioration, rates may fairly be predicated on the depreciated reproduction cost at the date of the valuation for rate-making purposes.

Although there is one notable exception which the writer will discuss later, the adoption of this rule will usually result in difficulties and confusion in accounting, with no attendant benefits either to the consumer or to the owner. Decreasing the amount allowed for interest and at the same time allowing a corresponding increase in amounts allowed to offset depreciation is a mere juggling of figures. An annuity sufficient to amortize the depreciated value of a property at the termination of its remaining life expectancy will become an "excessive" annuity on the day the first replacement is made, and generally the depreciated reproduction cost will be modified as a result of each replacement.

On the other hand, if we neglect the fluctuations in the prices of labor and materials, reproduction-cost-new will remain an unvarying quantity as long as no additions are made to the property, and the "adequate" annual depreciation reserve, estimated in the customary

Mr. Knox. way, will remain at all times a constant percentage of the unvarying reproduction-cost-new. The reproduction-cost-new having once been ascertained may subsequently be kept up to date merely by adding the cost of extensions as these are made, and the "adequate" annual depreciation reserve, having once been ascertained, may be kept up to date, as extensions are made, by adding sums sufficient to reproduce the cost of each such extension at the termination of its useful life.

If past depreciation reserves, with interest accretions, have been no more than adequate to offset the accrued depreciation on the date of the valuation, either method will result in justice both to the consumer and to the owner. The choice of method is merely a question of convenience, and generally it will be found most convenient to predicate rates on reproduction-cost-new, at the same time limiting the annual depreciation reserve to the "adequate" annuity, as previously defined.

The exception to which the writer has referred is presented by the case, briefly touched on by Mr. Alvord, and also discussed by Henry Floy, M. Am. Soc. C. E.,* in which certain large properties, after suffering an initial depreciation, reach a stable stage in which they may afterward be maintained by the expenditure of certain sums each year for replacements, the cost of these replacements being charged to operating expenses the same as ordinary wear and tear. To quote Mr. Floy:

"The Receiver of the Third Avenue Railroad in New York City, operating a large property having numerous physical elements so that all deterioration became simply 'wear and tear' and a part of operating expenses, declined to obey the order of the Public Service Commission and provided no depreciation fund whatever, simply removing deterioration when it occurred and charging it as maintenance in operating expenses."

If past depreciation reserves, with interest accretions, have been no more than "adequate" to offset "accrued depreciation," all subsequent depreciation on a property which has reached this stage of stability, may properly be taken care of in either of two ways:

1st.—By continuing the "adequate" annual depreciation reserve, in which case rates must be predicated on reproduction-cost-new.

2d.—By providing annually, in the manner adopted by the Receiver of the Third Avenue Railroad, a sum sufficient in itself to take care of all subsequent depreciation, in which case rates should be predicated on depreciated reproduction cost.

By the second method the annual depreciation reserve is in excess of an "adequate" reserve, as previously defined, and it is for this

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* "Valuation of Public Utility Properties," p. 194.

reason, and for this reason alone, that rates may be equitably adjusted to yield a fair return on a value less than reproduction-cost-new. Mr. Knox.

Consider, for example, a plant composed of five elements of equal value and variable life expectancy as shown in Table 4.

TABLE 4.

Element. (1)	Reproduction-cost-new. (2)	Life expectancy, in years. (3)	Annual depreciation reserve to refund reproduction-cost-new at termination of useful life, sinking-fund basis. (4)
A	$1 000	25	$24.01
B	1 000	30	17.83
C	1 000	35	13.58
D	1 000	40	10.52
E	1 000	45	8.26
Total, or average....	$5 000	....	$74.20

Assuming the scrap value to be nothing, the orthodox way of providing for the future depreciation on such a plant would be to set aside each year from the beginning of operation sums sufficient to amortize the reproduction-cost-new of each element at the end of its useful life. The annuities so required, estimated on a 4% sinking-fund basis, are given in Column 4 of Table 4. The total annuity required is $74.20. At the end of the twenty-fifth year, element *A* must be replaced, and the cost will be met by withdrawing from the depreciation fund the $1 000 accumulated by the annuity of $24.01 provided for this purpose; but, as the new element, *A*, will also have a life expectancy of twenty-five years, at the end of which it must again be replaced, the annuity of $24.01 provided to cover depreciation on element *A* must be continued and the total annuity will remain $74.20 as before. It will remain so indefinitely as long as no additions are made to the plant. Whether it be placed in bank or in trust at 4% interest, or invested in other ways, the annuity of $74.20 will, together with all the interest accretions with which it may fairly be credited, just take care of future depreciation, and no more.

Assuming that the owner actually makes such an annual reserve, the amount amortized at the end of the twentieth year will be $2 209.65, which amount may be taken to represent the accrued depreciation on that date. The depreciated reproduction cost at the end of the twentieth year will then be $5 000.00 — $2 209.65 = $2 790.35. As the owner can by hypothesis derive no "private gain" from the depreciation fund, however it may be invested, he must, so long as he provides for depreciation in this way, be allowed to earn a fair return on reproduction-cost-new.

Mr. Knox. Now assume that at the end of the twentieth year the owner perceives that 5 years hence he must replace element A at a cost of $1 000. He realizes further that at the end of each succeeding 5-year interval he must replace one element of his plant at a cost of $1 000. He estimates that the annuity necessary at 4% compound interest to produce $1 000 at the end of 5 years is $184.63, and he elects to take care of all subsequent depreciation by reserving this amount each year from earnings. The future cost to the consumer under this scheme will be increased in the annual sum of $184.63 — $74.20 = $110.43, and he must, obviously, if he is to avoid loss, obtain a reduction in the plant value or capitalization on which the owner is allowed to earn a fair return. Nor will the owner under these circumstances suffer any loss by such reduction in value. He has, in a sinking fund or otherwise invested, at the end of the twentieth year $2 209.65, equal to the accrued depreciation on his property as of that date. Under the first or orthodox method of providing for replacements, this sum, together with its earning power, was necessary to the proper provision for depreciation. Under the second scheme, neither the sum nor its earning power is so required. On the date on which the owner changes over from the first to the second method of providing for future depreciation, this fund is released or set free, and may henceforth be used productively by the owner in any way he sees fit. The consumer has in effect, by permitting the change in method, refunded to the owner, free and clear, a portion of his original capital equal in amount to the accrued depreciation on the property. Henceforth the owner's investment in plant is equal, not to the reproduction-cost-new, but to the depreciated reproduction cost, and this therefore is the sum on which he should be allowed to earn.

The case presented is only a special one under a general rule in which the interests of convenience are served by lowering below reproduction-cost-new the value on which the owner is allowed to earn a fair return, at the same time increasing above the "adequate" depreciation allowance, the sums annually set aside out of earnings to offset deterioration. It is in no sense an exception to the general law that where depreciation reserves are only "adequate" reserves, as previously defined, rates must be predicated on reproduction-cost-new.

The application of the general law will usually be more convenient, and will always result in justice both to the public and to the owners, unless we regard as an exception the case in which the owner has actually, during the past, set aside from earnings as an offset to deterioration an annual sum more than sufficient to take care of the depreciation which has occurred, or, what amounts to the same thing, unless the owner has actually during the past enjoyed excessive profits for which he is now to be penalized.

Mr. Knox.

A reduction in the value on which he is now permitted to earn may be logically justified on this ground only in the event that the present owner has benefited from these extortions. Even on this supposition, the logical and scientific procedure is to take as our point of departure the basic principle that in the absence of such complicating considerations, the owner should be allowed to earn on reproduction-cost-new; then to apply proper and specific allowances for such irregularities as the history of the plant may disclose. In this way "the punishment" may be made "to fit the crime," and we will always know, at least by how much and why, we have lowered or increased below or above reproduction-cost-new the value on which the owner is entitled to earn a fair return, for it must not be forgotten that it is a poor rule which does not work both ways, and if an owner is to be penalized for "excessive" depreciation reserves or exorbitant profits made in past years, consistency demands that he be compensated for his past failure to provide "adequate" depreciation reserves and fair profits.

If, for example, the owner in the case just discussed has, during the first 20 years, in which the necessity for replacements was not conspicuous, made no reservations whatever for depreciation, but has been satisfied to earn a fair return on his investment in addition to operating expenses, in the expectation that future replacements may be provided for as a part of operating expenses, when the occasion arises it is evident that his capital will be impaired by an amount ($2 209.65) equal to the accrued depreciation if rates are adjusted in the twentieth year to yield a fair return only on depreciated reproduction cost. In this instance, however, the loss is largely the result of his own folly, in failing to make provision at the proper time for the deterioration which was taking place.

He would be obliged to incur the same loss if he undertook to sell the property in the twentieth year, as it cannot be presumed that the physical elements of the plant could be sold on that date for an amount greater than their depreciated reproduction cost.

Mr. Humphreys, in discussing this question in the paper to which the writer has previously referred, quotes an amusing illustration taken from what he characterizes as the most logical paper on this subject he has ever read. The quotation is credited to a brief by Charles F. Mathewson, of the New York Bar, in the case of Kings County Lighting Company *vs.* The Public Service Commission for the First District of New York. Mr. Mathewson was the trial lawyer for the Company in the New York Consolidated Gas Company's case. The quotation follows:

"The proposition [to deduct 'accrued depreciation' in valuing plants in rate-making] is so absurd on its face that it hardly needs discussion to show its fallacy. Why, aside from the question of 'confiscation,' should consumers, for exactly the same service, equally efficiently ren-

Mr. Knox. dered, expect to pay less in the sixth year than in the first year, merely because some items of plant will (viewed at the sixth year) require replacement at a date in the future then nearer than such date was at the beginning of operation? As well might it be claimed, to repeat a homely illustration, that a farmer should regulate the price of the eggs which he sells, by the age of the hen which lays them—reducing the price of the product as the hen gets on in years. The reason he does not is that the service efficiency and operating value of the hen, as evidenced by the quality of the eggs which she lays, are not impaired by the fact that her life is advancing. That advancement may concern the farmer and possibly concerns the hen; but it in no manner affects the value of the eggs to the consumer, or justifies him in demanding them at a lower price than he paid at an earlier period of her life. The consumer of the eggs must expect to pay a sufficient price to afford a return to the farmer on his total investment in the hen during her life, *plus* enough more to enable the farmer on her death to replace her, and thus keep his investment unimpaired. A farmer could hardly be expected to invest in hens for the purpose of supplying the public with eggs, if for a portion of their life he was to receive a return on only a third or a half of his investment; and any such rule would simply compel the public to go without eggs until the regulating power (if such there were) saw fit to revise its reasoning. There is absolutely no difference in the economic principles applicable to the operation of a gas plant and the operation of a hennery, so far as concerns right to return on capital; and what is absurd in one case is equally absurd in the other. The fact that the rate of return in the one case is subject to reasonable regulation, and not in the other case, has no bearing on the main proposition."

A consideration of this illustration in the light of the principles thus far set forth indicates that it is not of universal application except as an illustration of the fundamental principle which always holds where complicating considerations are absent. It presumes that the farmer charges no more than enough to afford a return on his total investment plus enough to keep his investment unimpaired. If, as some consumers think, the farmer charges enough in addition to a fair return on capital invested, to replace the hen several times over, prior to her demise, the absurdity of expecting a reduction in the price of eggs, even in this case, from a purely theoretical point of view, is not quite so apparent.

It is readily admitted that the preceding discussion has been carried on largely from a theoretical point of view; but, considering the present confusion of ideas on the subject, this theoretical treatment of the question appears to be fully justified. Sound practice, in this as in other engineering matters, must rest on a firm foundation of sound theory, and the establishment of this theory has something more than academic interest.

In concluding it may be well to point out that too much importance may easily be attached to the fact that the Supreme Court of the United States has in certain instances held that rates should be ad-

justed to yield a fair return only on depreciated reproduction cost. It is of course true that the last word in these matters rests with the Courts, and that Courts are much given to following precedent; but it must not be forgotten that Court decisions are usually rendered with a view to dispensing justice in the particular case before the Court. It has been shown that there may be circumstances in which the ends of justice are served by allowing an owner to earn a fair return on depreciated reproduction cost. Whether or not the cases so decided by the United States Supreme Court fall in this category we do not presume to say, but even this is unimportant because it cannot be presumed that the Courts will seek to perpetuate what can be demonstrated to be a wrong merely to avoid going against their own precedents. The question cannot be regarded as finally settled until it is settled equitably, and when, as in the present case, such a wide diversity of opinion prevails among those who have spoken most authoritatively on the subject, the attempt to reason the problem out *a priori* seems to be fully justified. Mr. Knox.

J. H. Gandolfo, Assoc. M. Am. Soc. C. E. (by letter).—In a discussion dealing with appraisal, valuation, depreciation, rate-making, or kindred subjects, the argument is frequently advanced that no consideration must be given in cases in which all matters pertaining thereto have not been conducted in a fair, honest, proper, and business-like manner. If the human race was still living in the Garden of Eden; if Pandora had never allowed all those troubles to escape into the world; if there were no such things as selfishness, greed, hate, envy, revenge, and a host of other evil passions; in short, if the great struggle for existence throughout the world (and this struggle applies to all plant and animal life, as well as to all members of the human race) did not exist, then it would be possible to discuss such questions purely academically, and arrive at such a course of conduct for all parties concerned as would lead the human race along a broad and happy highway of peace, prosperity, and contentment. Under these circumstances, the very necessity for such things as public service commissions, appraisals, hearings, judicial reviews, and hosts of kindred matters would not exist. Mr. Gandolfo.

Unfortunately, however, the millennium is still as far away as ever. It is just such elements as those described in the first part of the foregoing paragraph that go to make up the human mind, and therefore such elements must be taken into consideration and given careful attention in such discussion. In other words, it is the much used term "human nature" that must be studied and reckoned with, and taken into account, in such investigations. Facts on record relating to bond and stock issues, and to general methods of financing and of business, especially in the railroad world, have made history, showing most con-

Mr. Gandolfo. clusively that human cupidity and the still baser human passions have played a most important rôle in these matters from the year 1838 to the present day, resulting in the loss of millions of dollars to honest investors, both large and small.

Mr. Alvord says, on page 791: "In other words, no amount of accounting or theorizing can make good a depreciation fund which is actually not in hand." In the succeeding pages, however, he seems to lean to the idea that a sinking fund provided to take care of depreciation or obsolescence can be invested in some other enterprise and still remain a sinking fund, and goes into a great deal of detail to explain why this is so. For example, on page 799, he says: "* * * for instance, such monies may be invested as new capital in new construction and extensions in the very plant itself to great advantage * * *."

Webster defines "sinking fund" as follows: "*(Finance)*, a fund created for sinking or paying a public debt, or purchasing the stock for the government." The Century Dictionary and Cyclopædia gives the following: "A fund formed by a government or corporation for the gradual 'sinking', wiping out, or reduction of its debt, by various devices for the accumulation of money." Funk and Wagnalls' Standard Dictionary says: "A fund instituted and invested in such wise that its gradual accumulations will enable it to meet and wipe out a debt at maturity."

In view, therefore, of the very fundamental idea and *raison d'être* of a sinking fund to cover depreciation and obsolescence, it is an absolute impossibility to consider it as being invested in any other enterprise, except as a bank or trust company would invest such funds. The moment such a sinking fund is used by the owner of a plant or a board of direction for any purpose other than that for which it was originally set aside, the fund, as a sinking fund, ceases absolutely to exist. No longer can it be counted on as needed to replace the plant. It is gone, absolutely and entirely, as far as its original purpose was intended, and no amount of bookkeeping or argument can make anything else out of this fact.

On page 797 Mr. Alvord says: "* * * there would be no difficulty or hazard in relying on the ability of the owner or owners to replace the sinking fund from outside sources * * *." On page 798 he also says, "It may be objected that it is a hardship on owners of public utilities, who are able and willing to replace in the property such portion of the renewal fund at any moment * * *," and on page 799 "* * * the owner must be willing and ready to finance the replacements and depreciation fund, whenever the occasion therefor arises * * *." Admitting for the moment the willingness, suppose, when the time comes, that the owner is not able to do so? What then? And further, suppose that such a fund has been invested in some

other way, or has been appropriated for some other purpose, and then the parties at fault, although fully able to do so, are not willing to make restitution when the fund is needed for its original purpose. Who is to make them do so? Mr. Gandolfo.

In connection with this matter of the investment of a sinking fund, it may be argued that the bank or trust company in which such a fund would be deposited, to draw interest at ordinary rates, would invest these moneys in some way, and therefore there is practically no difference in the end, than if the owners invested the fund direct. There is a very great difference, however, which may be summed up as follows:

(1) The owner is an interested party, and as such, is not capable of investing the fund to as good advantage as the bank. He would be more apt to have bias or prejudice, good or bad, and it would be absolutely impossible for him to look at things in the free, untrammeled way that the officers and board of direction of a bank are supposed to do.

(2) The fund in question is not the only resource possessed by the bank. There are dozens of others, including capital and surplus, and if one fails or is in such form that it cannot be realized on at once, there are others that can be. In this way, the bank may be likened to an equalizing pond or reservoir for the benefit of all its depositors. The drain in one direction may be heavy, while that in others is light, and at the same time there is a constant supply from all sources. This condition could not possibly exist if the fund was directly invested by the owner.

(3) The bank or trust company is in its turn a "Public Service Utility", subject to the most careful scrutiny by the State, and thus an additional safeguard is thrown about all sums placed here in trust. If the fund was privately invested by the owner, there would be no such safeguards.

Further, if the owner, as stated by the author, must always be ready to make good the depreciation and obsolescence, he in his turn must have funds immediately available with which to do so. As, following out this line of argument, he would ordinarily have all his personal funds invested, it would then be necessary for him to maintain some fund for this particular purpose. Wherein lies the difference, then, under such conditions, whether such a fund is held by the original party, or must go through several hands, except the far greater risk and uncertainty in regard to its ever materializing at its original source when needed?

As far as investing a sinking fund in extensions to the plant, the moment it is so invested, it becomes "Capital", subject at once in its turn to depreciation, and is no longer "Sinking Fund". Further, if the same rate of profit is allowed on the sinking fund as on the plant (and, under conditions as hereinafter described, there seems to be no

Mr. Gandolfo. question but that there should be), then there is no difference as to whether the fund is invested in new construction or not, as far as the returns to the owner are concerned.

As for a concrete example illustrating much of the foregoing, there is one in the New York, New Haven and Hartford Railroad. The funds of this road, which should have been used to take care of depreciation, obsolescence, and improvement, within the last few years have been used for other purposes, the details of which it is needless to go into here. When, a short time ago, on account of a series of accidents that cost the lives of scores of people, an investigation by the State disclosed the fact, among others, that sections of this road were still equipped with signals of a type which had been discarded as obsolete by other roads some fifteen or more years ago, and it therefore became imperative to use for improvements the funds that had been otherwise "invested", did the parties who had appropriated them make any move to replace them? When public opinion, however, at last aroused, demanded that something be done, $67 552 000 of 6% convertible debenture bonds were offered to the general public, in order to take care of some $46 023 750 of obligations which are to mature within comparatively short periods, the remainder being to pay for financing and to obtain funds to make good the depreciation and obsolescence. The issue of these bonds has been opposed by the present bondholders, and the offering of them is now awaiting the decision of the Court. In place of this, approximately $45 000 000 of short-term notes were issued, to pay for the immediately maturing obligations, and also to provide for pressing needs due to depreciation and obsolescence.

Further examples, such as the foregoing, might be given in detail, but there is very little use or profit to be gained by anybody in arguing theoretically about a question of this kind, when undeniable facts and occurrences are on record before the financial world and the general public.

As the author points out, at no time should a depreciation and obsolescence fund amount to a very large percentage of the cost of the plant, on account of the constant withdrawals from it to maintain the plant in first-class and up-to-date condition. If such a fund is actually in hand, that is, actually in trust, there is no question but that rates should be allowed on it, just the same as on any other item of the plant. If the plant is being kept in repair and up to date by constant withdrawals from the fund, there seems to be no reason that rates should not be allowed on the full invested capital of the plant. This matter can be treated in two ways, which lead to the same result: The property can be appraised at its full cost, without depreciation, and returns can be allowed on this figure, no special mention being made of the fund; or the property may be appraised at its cost, less depreciation, and returns may be allowed on this, and also on the

depreciation fund. Of course, it is assumed that the sinking fund is always approximately equal to the actual depreciation at any time. A sinking fund, provided for depreciation and obsolescence, and actually in hand, is part of the plant, and should be treated as such. Mr. Gandolfo.

The necessity of keeping a public utility property in first-class condition by constant withdrawals from the sinking fund can be further illustrated by the following theoretical case. Suppose that the life of the physical property has been determined, and then a depreciation fund is created which, at the end of the period, will just return 100% on the investment. At the end of this period, the plant, like the deacon's famous "one-hoss shay", would theoretically vanish into thin air, leaving the sinking fund in its place. If a fund under such conditions was allowed to accumulate, then the theory that a public service utility must render adequate and constant service to the public is untenable, and cannot be upheld, as the owners would have a perfect right to allow the plant to deteriorate gradually to nothing, or to wipe out the utility at such time as they saw fit, and leave the public without its benefits.

This question of constantly keeping up the depreciation of a property is referred to in Knoxville *v.* Knoxville Water Co., 212 U. S., 1, Jan. 4, 1909, which says that the Company "* * * is entitled to see that from earnings the value of the property invested is kept unimpaired, so that at the end of any given term of years, the original investment remains as it was at the beginning". That public service corporations are expected to keep the plant up to date, without allowing any excessive accumulation of a sinking fund, is shown by the opinion of Judge Evans in Cumberland Telephone and Telegraph Co. *v.* City of Louisville, 187 Fed., 637, April 25, 1911, and that of Judge Haight in People *ex rel.* Manhattan Railway Company *v.* Woodbury, 203 N. Y., 231, Oct. 17, 1911.

On page 798, Mr. Alvord argues that if a sinking fund has been withdrawn from the plant and used for other purposes—or, as he puts it, for private gain—then the owners are not entitled to rates on it. The writer fails to see why it is necessary to argue on such a point as this, as it is a self-evident fact. If rates were to be allowed on something in the way of a sinking fund that does not exist, then it would be just as logical to remove an entire battery of boilers (or any other part of a plant), erect and use them in some other place, and then claim rates on their value, on the ground that the owners would replace them if needed. Thus the argument might be continued *ad infinitum* until the best part of a plant had been removed and rates were still being allowed on it as originally installed.

On page 803, under the 5th conclusion, the author says: "That though it is not now considered improper to use reserve or sinking funds allowed for depreciation for private gain * * *". The writer

Mr. Gandolfo. hardly thinks that Mr. Alvord can have realized what he was saying when he made this statement, or else that he did not mean what he says. If a private individual owns a plant himself, and sets aside a sinking fund for depreciation, and then after a time withdraws this fund and uses it for some other purpose of his own, but still calculates that he has a depreciation fund, he simply deceives himself—and there is no deception so fatal as self-deception. If, on the other hand, the concern is a stock company, and a sinking fund for depreciation and obsolescence has been created, and then the officers, directors, or managers should use this fund for their own private gain, it would amount to misrepresentation and fraud to both stock and bond holders on the one hand, and misappropriation and embezzlement of funds on the other, both of which are criminal offenses under our penal code. Furthermore, even if a board of direction did use such a fund in some other way, even if for the direct benefit of the stockholders, it is very doubtful if, under a strict code of ethics, it would have a moral right to do so, without first having obtained the majority vote of the stockholders to such a venture; although, in such a case, the strict letter of the law would doubtless uphold a board in such a proceeding.

In conclusion, it may be said, therefore, that a sinking fund to cover depreciation and obsolescence must be kept constantly in hand, and be available at all times; that constant withdrawals must be made from this fund to keep the plant in first-class and up-to-date condition (that is, 100% efficiency and value); that such a fund cannot be invested in other enterprises, as then it is no longer available for its purpose, and does not exist; and that, unless such a fund, or its equivalent in some form or other, exists, the plant is not entitled to full cost or replacement value, but must be appraised on a basis of cost or replacement value less depreciation.

Mr. Harte. CHARLES RUFUS HARTE, M. AM. SOC. C. E.—Papers such as Mr. Alvord's, which by stimulating discussion tend to spread a broader knowledge of the important subject of valuation, are particularly valuable at this time when the situation seems not unlike that which led the old New Englander, not in sympathy with the party then in power at Washington, to add to his prayer for Divine guidance for the then President the earnest statement "Thou knowest, Lord, he needs it." At the same time, it would seem not entirely out of place here to raise the question whether, in the treatment of the subject, many of us are not beclouding the facts by analyses and discussions which are too academic.

The author's definition of "depreciation" includes four widely diverse items of loss of value:

1.—Organic depreciation—the loss of value because of wear of the substance of the elements which make up the machinery or plant—begins with the life of the plant and continues practically uniform to

the end. Plotted in terms of time and loss, it is very close to a straight line. **Mr. Harte.**

2.—Functional depreciation—the loss of value because elements of the plant get out of correlation—begins with the life of the plant, and, particularly if not met by proper maintenance, accelerates very rapidly until near the break-down point, when it checks and then continues slowly to the end. Proper maintenance brings back conditions, but not quite to what they were at the previous take up, so that, plotted in terms of time and loss, functional depreciation appears as a saw-toothed curve.

3.—Obsolescence—the loss of value because the product of the plant no longer meets market requirements as to character—may attack a property at any time, or may not occur at all. Occurring, it may progress with great rapidity; it may progress slowly; or it may progress, halt, and progress again; so that its curve, in terms of time and loss, may be of almost any form. A striking instance of acute obsolescence is that which, almost over night, changed the cable traction system of the Broadway street-car line in New York City from the latest word in traction to a practice not merely obsolescent but actually obsolete. Thus, also, the rapid development of the alternating-current lighting system with high-potential primaries feeding through transformers, the low-potential distribution circuits, in an almost equally brief time made the Edison three-wire underground system practically of the past.

4.—Inadequacy—the loss of value because the product of the plant, though meeting the requirements as to character, is of insufficient quantity—is, like obsolescence, uncertain as to its occurrence and variable as to progress, though not to the same extent as the former; and, like it, its curve, in terms of time and loss, may be of almost any form.

These four elements are so different in character that it is hardly possible to treat them properly together. If the system under consideration is of such extent that the average of conditions is a fair general average, life tables may be applied with very good results, so far as organic and functional depreciations are concerned. Local conditions, however, often make the system average quite different from that of the tables, with corresponding error unless proper correction factors are used. Obsolescence and inadequacy, at best, can only be guessed at.

The author truly says of "the reproduction method, or cost new less depreciation," "This line of evidence is only one source of information as to value." The fair value of the property used and useful for the public is neither the cost to produce nor yet the cost to reproduce; it is the capitalized earning power. It has been argued that for rate-making the earning power cannot be considered, as it is a direct function of the rates to be regulated, and it is hard to escape

Mr. Harte. the conclusion that—at least where there is competition—rate-making must of necessity be an arbitrary procedure. Economic laws, more powerful even than governmental commissions, will compel the practical equalization of rates, even though different values of competing utilities would otherwise permit them to charge different rates for the same service.

In determining physical value, the Courts almost universally—if not entirely so—have held that depreciation must be considered. It is difficult to find logical grounds for doing otherwise. Use—and for that matter disuse as well—causes losses of quality, quantity, and, if extended, of efficiency. That these losses ought to be made good by the owner, or that there exists a fund dedicated to such making good, in no way changes the facts that the loss has occurred, although such outside conditions may offset the physical loss and result in an unchanged or even enhanced value when the two are considered together.

With the proposition to determine past depreciation "on the basis of a sinking fund" there will be much disagreement. Past depreciation is a fact which, except as to its rate of progress, is usually ascertainable without great difficulty, and it should be actually ascertained if the determination is to stand investigation.

A depreciation fund properly secured may be included as one of the physical parts of a property, and, if equal to the amount of depreciation, will, of course, offset it, but in such case the depreciation itself must be determined in order to see if the fund is adequate; it is equally possible, unless adjusted frequently, that it may be greater or less. Nor is the writing off of such an item on the books without dedication of the actual money merely an opinion. Properly written off, it stands as a liability of the property, reducing by its amounts the net earnings of the prosperous plant, or adding to the deficit of the losing venture.

Under the caption "Utilities Commonly Require Growing Plants" the author states that extensions produce a developing series of increments, each with its own depreciation. Although this is literally true, the situation is far less complex than the statement would indicate, because much of an extension merges into and thereafter is treated as a part of the older installation, and depreciation is taken as an average on the whole. Building extensions of comparatively small amount, additions to piping and wiring systems, all such are adaptable to this treatment, leaving only the larger individual items to be dealt with separately.

It might well have been said that the earning power of a partly worn plant is almost invariably greater than that of a new one. Smooth operation, and the maximum efficiency, are had only after the very appreciable physical losses due to what Mr. Wilgus, in his recent paper

"Physical Valuation of Railroads,"* aptly called the "educational period." Mr. Harte.

The author states that "One of the reasons the Courts have for favoring the reproduction method, or cost new less depreciation, lies in the fact that such procedure includes appreciation as well as excludes depreciation." It is probable that the attitude of the Courts in this matter is due more to the logic of the procedure than to a desire to include appreciation.

The cost of educating the public to the value of the utility, though not attaching to the property in that particular form, remains as a very considerable asset in the going value, which is due to "a fundamental conviction that a utility is really necessary and useful."

In "The Practical Treatment of Depreciation Accounts" the author apparently considers safe only that fund established with some trust company. Aside from the fact that trust companies have failed, and doubtless will do so in the future, such companies find the necessary monies to meet fund interest, overhead charges, and profit, in other securities, and, if the utility by direct investment can save the middleman's profit and overhead charges, and thus reduce the investment necessary to produce the required result, it would seem but fair thus to permit; the matter is purely the question of fact as to whether or not the "securities" are such in more than name, and whether they are a part of the property, or have passed out of it. The first calls for expert financial knowledge; the second must be evident from the books.

A reasonably safeguarded fund or investment for renewal purposes, in a total valuation, should be considered as much a part of the property as the material which it will presently replace. Such replacements for the most part can be made economically only after considerable wear and consequent loss in physical value. If this physical loss is not excessive, and is properly set up on the books as a liability, it is obviously unjust to deprive the owner of its proportion of earnings merely because of its changed form.

The author's first conclusion, that a trust fund to meet depreciation if "properly computed and accurately kept"—this whether in this connection "kept" has reference to the bookkeeping or the actual holding—should be considered a part of the property "used and useful for the public," would seem obvious.

The second conclusion, that the fund must be "actually in hand," is also obvious, if a reasonable interpretation is put upon "in hand." To say, however, that proper book entries do not constitute such condition is, to put it mildly, unfair; carried to its logical conclusion, such reasoning would preclude the consideration, as assets, of any unpaid accounts.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 208.

Mr. Harte.

The third conclusion, that "past depreciation may be computed accurately on the sinking-fund basis," is only warranted when "all the facts of the past are known," in which instance there would seem to be little occasion to compute what is already had. It should not be forgotten that, in the Minnesota Rate Case, there were thrown out of Court, because obtained by hypothetical assumptions, figures for land values which were undoubtedly very close to the facts. For the future, knowledge of the past will often establish correction factors to apply to and make applicable life tables, so that organic and functional depreciation can be predicted with reasonable closeness, but the possible effects of obsolescence and inadequacy can only be guessed at, in the light, respectively, of the development of the art and the local conditions.

Conclusion four, that depreciation accounts should be kept by groups having similar life characteristics, and should be revised from time to time, is good, and might well call for considerable detail in the groups. It should be always remembered, however, that such accounts are only estimates, the relations of which to the facts can only be known by actual comparison.

Conclusion five, as stated, that the use of a reserve or sinking fund for private gain precludes its consideration as a factor of value to the utility, is sound only as it applies to such portion of the return from the fund as becomes "private gain." The use of funds set aside for the specific purpose of a reserve or sinking fund in the purchase of securities or in other forms of loan does not destroy the asset unless the security therefor, whether bonds, stocks, or personal notes, loses value; nor if such security pays a rate of interest higher than necessary for the calculated fund, and this excess be withdrawn and otherwise used, does it impair the protection of the remaining portion.

Mr. Kiersted.

W. KIERSTED, M. AM. SOC. C. E. (by letter).—Mr. Alvord's interesting paper on depreciation appears to be largely a discussion of the paper by Mr. Grunsky,* or at least to have been suggested by the views set forth in that paper.

In this discussion Mr. Alvord raises four distinct questions, namely:

Shall allowance be made for depreciation of physical structures in computing value of a public utility property by the reproduction method for rate-making?

Should a sinking fund, created to take care of depreciation, be allowed to earn a return—in fact, to earn the same rate of return as other portions of the property?

Should a sinking fund thus created be considered as a part of the property?

Should the units of property be grouped according to their respective life limit in accounting?

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* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 770.

A difficulty at once arises in discussing these questions, from the fact that the viewpoint of the author is not made perfectly clear. One can readily conceive of lines of procedure under the reproduction method of computing value which might admit of opposite answers to all the questions mentioned. Under the circumstances, the discussion of these questions is likely to develop divergent opinions, depending on the viewpoint of the writer. Mr. Kiersted.

The reproduction method of valuation seems to require more hammering to reduce it to proper form and more clearly to restrict or fix the line of procedure; and, until this is done, there is little hope of arriving at a conclusion on questions of the kind raised by the author, especially when considered somewhat abstractly. In many instances computations relating to the rate of return on values found by the reproduction method have not sufficiently discriminated between a fair rate of return on the value of the depreciable and the non-depreciable property, on the value of the tangible and the intangible property, and on the value which is actually earned and that which includes an unearned increment. Such questions as these seem to have been overlooked in most of the rate cases of recent years. Moreover, most of the units going to make up the value of a public utility property possess no value except for the particular purpose for which the public utility is used, though other units possess an intrinsic value of their own, entirely apart from and independent of the public utility with which they are connected. Investment in units of one class is obviously more hazardous than in the other class, and there should be a corresponding difference in the rate of return to which the respective classes of property may be entitled. The questions raised by the author fall in a similar category, and unless they can be discussed thoroughly and comprehensively in connection with all other questions which enter into the computations of value by the reproduction method, or by any other method, no conclusive answer can be given to them.

The writer's discussion will be brief, and directed largely to simplifying, rather than complicating, the questions propounded for discussion.

Several valuations of water-works properties by commissions of experienced men, for the purpose, among other things, of rate-making, have been carefully analyzed and platted for the purpose of seeing how the methods of valuation therein used would work out in practice. However, only that part of one of the analyses which seems to apply to this paper, will be considered. The value of this particular property was computed by the commission in accordance with the reproduction method, as used in many of the late valuation cases, and allowance was made for depreciation as well as for enhancement of value and

Mr. Kiersted. for unearned increments of value. Depreciation was computed on the sinking-fund basis by assuming a life for thirty or more units going to make up the physical property, with sinking-fund increments uniformly bearing interest at the rate of 3½% per annum, regardless of whether the unit under consideration was short-lived or long-lived. Under the assumptions made in the writer's analysis, the composite property might or might not be assumed to have a definite life, but, for the purpose of a part at least of this discussion, it is considered as possessing definite life—a life limit having been assumed for all the units of the depreciable property.

On the diagram, Fig. 1, are platted two curves, each point in each curve being platted with reference to the average age and equivalent annual percentage rate of depreciation of the composite property. One, the full-line (upper) curve, is for convenience termed a definite-life curve, the other, the broken-line (lower) curve, is termed a perpetual-life curve. Average age is computed in the usual manner by the weighted method. The equivalent annual percentage rate of depreciation is found by dividing the total depreciation, as of the date under consideration, by the average age of the composite property, as of the same date. Whenever a break occurs in either of these curves it indicates that some one of the many units considered has reached the limit of its assumed life. It is assumed that throughout the whole period under consideration the total investment in the physical property remains the same. This assumption is met in the analysis by making two computations whenever any unit, going to make up the composite property, reaches its life limit. One computation consists of determining the average age and the total percentage depreciation of the composite property at the time when a unit reaches its life limit and is depreciated 100%; the other computation consists of determining the average age and total percentage depreciation as of the same year and date as the preceding computation, taking into consideration the new or substitute unit as of the same class and cost as the replaced unit. The new curve, starting with the substitute unit, continues unbroken until a second unit reaches its life limit, when another break occurs and two similar computations are made and a new curve started, and so on until all the units have been thus considered, making what is for convenience termed a cycle.

During a cycle of this particular property the number of units replaced by new ones of the same kind and cost are as follows: four units four times, eight units three times, nine units twice, and ten units once.

These curves serve no particular purpose except to illustrate the variation of the equivalent annual percentage rate of depreciation of the composite property during different periods of a cycle. They

Mr. Kiersted

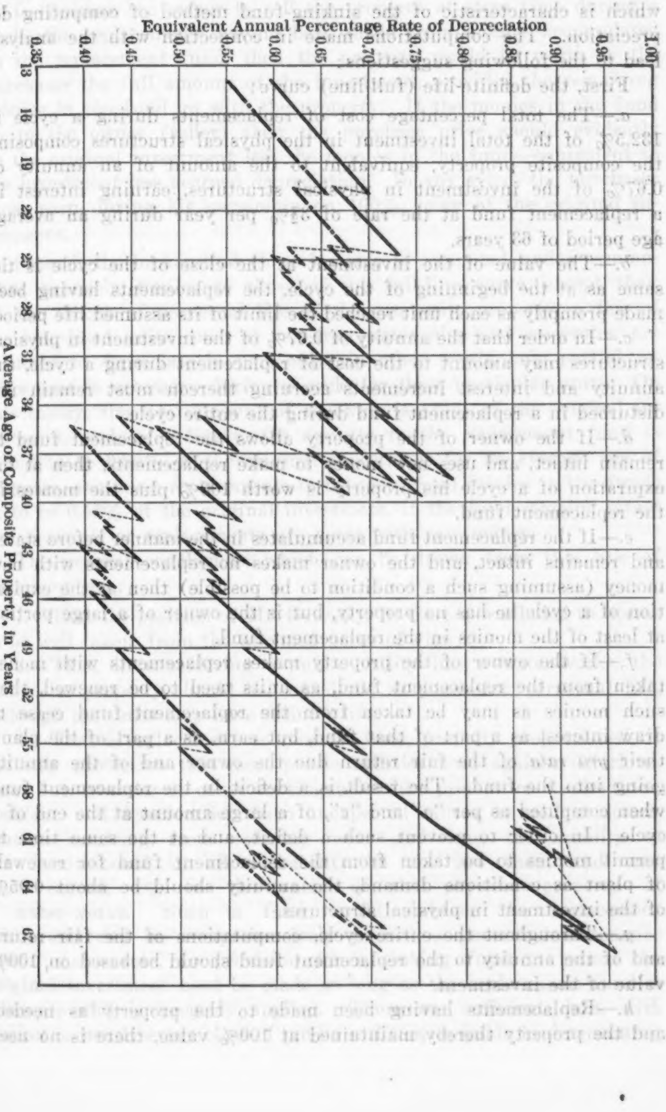

Fig. 1.

Mr. Kiersted. indicate a progressive and regular increase of the equivalent average annual percentage rate of depreciation until units begin to disappear, which is characteristic of the sinking-fund method of computing depreciation. The computations made in connection with the analysis lead to the following suggestions:

First, the definite-life (full-line) curve:

a.—The total percentage cost of replacements during a cycle is 132.5% of the total investment in the physical structures composing the composite property, equivalent to the amount of an annuity of 0.67% of the investment in physical structures, earning interest in a replacement fund at the rate of 3½% per year during an average age period of 63 years.

b.—The value of the investment at the close of the cycle is the same as at the beginning of the cycle, the replacements having been made promptly as each unit reached the limit of its assumed life period.

c.—In order that the annuity of 0.67% of the investment in physical structures may amount to the cost of replacement during a cycle, the annuity and interest increments accruing thereon must remain undisturbed in a replacement fund during the entire cycle.

d.—If the owner of the property allows the replacement fund to remain intact, and uses new money to make replacements, then at the expiration of a cycle his property is worth 100% plus the monies in the replacement fund.

e.—If the replacement fund accumulates in the manner before stated and remains intact, and the owner makes no replacements with new money (assuming such a condition to be possible) then at the expiration of a cycle he has no property, but is the owner of a large portion at least of the monies in the replacement fund.

f.—If the owner of the property makes replacements with money taken from the replacement fund, as units need to be renewed, then such monies as may be taken from the replacement fund cease to draw interest as a part of that fund, but earn, as a part of the plant, their *pro rata* of the fair return due the owner and of the annuity going into the fund. The result is a deficit in the replacement fund when computed as per "*a*" and "*c*", of a large amount at the end of a cycle. In order to prevent such a deficit, and at the same time to permit monies to be taken from the replacement fund for renewals of plant as conditions demand, the annuity should be about 0.95% of the investment in physical structures.

g.—Throughout the entire cycle, computations of the fair return and of the annuity to the replacement fund should be based on 100% value of the investment.

h.—Replacements having been made to the property as needed, and the property thereby maintained at 100% value, there is no need

of a sinking fund to return the original investment, because the property possesses the characteristics of perpetual life. Mr. Kiersted.

i.—Assume the owner to sell the property at some time during a cycle, the replacements having been made from monies accumulated in the replacement fund, then the property would be worth to the purchaser the full amount of the investment, provided the remaining monies in the fund go with the property. If the monies in the fund go to the owner (seller) then the purchase price should evidently be the original investment less the monies in the fund—equivalent to the investment less accrued depreciation. In any event, the purchaser must earn during his ownership on 100% value of the original investment.

Second, the broken-line (lower) curve is computed under assumptions precisely similar to those of the full-line curve, except that, on the theory of perpetual life of the composite property, a few of the long-lived units, such as cast-iron water pipe and reservoirs, are assumed to depreciate in part only; in other words, only partial replacement is necessary during a cycle for these particular units. On this theory, the total amount of renewals is 114% of the original investment, and, accordingly, the annuity to the replacement fund is found to be 0.80% of the original investment for an average age of 65 years, assuming replacements to be made from monies in the fund; or to be 0.53% of the original investment, if the fund remains intact and renewals or replacements are made with new money.

The general conclusions which can be drawn from this line of discussion are:

1.—That the position of Mr. Grunsky, in the paper alluded to, seems to be well taken, from the assumed point of view.

2.—That, in considering a public utility like a water-works, the composite property should be considered as possessing perpetual life, although the units may have definite life.

3.—That a replacement or renewal fund, properly proportioned, is the only protection that most public utilities need against deterioration; but, in order to cover contingencies like premature obsolescence, accident, and deterioration from unexpected causes, as well as deterioration resulting from ordinary wear and tear, the annuity to the fund should in all probability be not less than, and may even exceed, 1% of the value of the physical property of a water-works. Such a fund would embrace the replacement fund, and can be comprehensively termed a general maintenance fund. As a rule, no provision for a sinking fund to return the original investment need be made so long as the replacement fund is adequate and is properly applied. There are sufficient data, with respect to nearly every class of public utilities, to guide in determining

Mr. Kiersted. an equivalent percentage rate of deterioration of physical structures by which to compute the annuity to a replacement fund. The annuity itself will doubtless vary for different properties of any given class. The annuity to a fund should be considered as earning interest at sinking-fund rates, the interest being considered, of course, as a part of the fund. None of the increments to a replacement fund should be capitalized. The monies in the fund should be used as needed for replacements, and should not be surrounded by a wall which prevents their proper application in that direction.

4.—That the question of whether a replacement fund shall or shall not earn a return is not one for consideration by a public utility commission in definite detail. The office of such a commission is to see that there is a fair return for rendering good service, and that the properties are maintained in such condition and so operated as to serve the public properly. The question whether the monies in a replacement fund should be considered as a part of the property for use only in replacements or to go to the owner of the property, is a debatable and important one. Should it go to the owner, he is obliged to credit the fund with the regular sinking-fund interest increments, but it cannot be conceived that any public utility commission would seek to destroy the incentive to good business management by ruling against the earning of a return on monies in such a fund, so long as replacements with full-value units are made promptly as needed; nor should such a commission delay in calling to account a management which ignores the public interest by delaying replacements when needed, in order to earn a return on the monies in a replacement fund. If the fund remains attached to the property and is to be used for replacements only, the annuity must be proportioned with some liberality, in order to avoid, if possible, a deficit; but if a deficit occurs and new money goes into replacements, then the owner is entitled to a re-rating of the property, a return of the value of his new-money replacements, or an increase of capital investment. On the other hand, the public is entitled to similar consideration under reverse conditions. In other words, properties must be re-rated periodically.

5.—That depreciation should be considered in connection with the valuation of the physical structures of public utilities on the reproduction basis. Depreciation may or may not be deducted from the value thus found, depending on the manner of the application of monies in the replacement fund and the extent to which the unearned increment enters into consideration. If replacements have been promptly made, and the policy of the management is to maintain high plant efficiency and good public service, there should be no depreciation of original cost value of existing physical property in a rate case; but, where reproduction forms the basis of value for rate-

making, and such value includes an unearned increment, due to changed physical conditions or enhanced values of certain units resulting from city environment, and the same rate of return is allowed on these elements of value as on other portions of the property, then depreciation should be allowed in computing fair return; but if the enhanced values and unearned increments of value were put in a class by themselves and were allowed such a rate of return as may be equitable, then depreciation of the physical property might be as consistently ignored in a rate case based on reproduction as when based on original cost value. Mr. Kiersted.

6.—That, in accounting, there is no necessity for dividing a property into groups of units according to some assumed life and charging each account with depreciation in accordance with some mathematical rule. The property should be considered as a whole, or in groups arranged by structural classification, deducting from the values thus recorded in the accounts the cost or value of any article or unit that is abandoned or replaced, and adding to the account the substitute article or unit. In doing so, proper discrimination should be drawn between minor renewals belonging properly to the operating account and replacements which actually affect the investment. Every engineer must concede that the place for working out details and for clarifying and simplifying any plan or specification is in the office, in order to render the work of the field parties as simple, clear-cut, and free from the danger of error as possible, and it can be conceived that similar principles should apply to accounting.

Although much more can be said on the questions under discussion, if considered in their relation with the general subject of valuation, and from a practical rather than an academic point of view, the writer is not inclined to extend the range of discussion beyond that laid down in the paper, except to add one or two remarks, particularly as the writer's discussion of Mr. Grunsky's paper presents relevant views which Mr. Alvord seems to indorse, in part, at least.

The writer has stated previously that the reproduction method of valuation needs more hammering to reduce it to proper form for general use. The attempt to apply rules to valuation work, with mathematical precision, is certainly a mistake, and leads the adherents of these rules into error in matters which necessarily involve the use of an enlightened and experienced judgment unfettered by precedent or rule. So many times the "yard-stick" of value (to use the term of another) in contact with a mind overheated with the discovery of some element of value hitherto disregarded, or with some new idea, has been so expanded as to distort a line of reasoning believed at the time to proceed from an elevated or progressive point of view; or, on the other hand, the "yard-stick," with the middle half cut out and the ends

Mr. Kiersted.

of the remaining portion joined with unaltered dimension at either end, has been used as though of up-to-standard length. There is no possible way of reconciling the differences thus encountered, or of bridging the wide gap exposed in the deductions thus made, except by going back and standardizing the "yard-stick." In other words, fundamental principles must be agreed on, and these principles must be applied consistently, intelligently, and cautiously, and with due consideration of the character, condition, and individuality of each property to be valued. In no two instances will the application of the principles be the same, but the results of the work of two or more appraisers should not vary so much as to be irreconcilable, or as to cause confusion of mind for those who hope to be profited by the expert's wisdom.

Mr. Stone.

GEORGE B. STONE, ASSOC. M. AM. SOC. C. E. (by letter).—The question of depreciation of public utility corporations is of great moment at the present time to all members of the Profession, and especially to engineers engaged in the work of valuation of public service corporations, and is a subject of vital importance to both the public and the financial interests. Mr. Alvord's paper is of great interest, and he has presented his view of the question clearly and concisely. To the broad discussion that must take place before we finally come to an agreement, his paper is a valuable contribution.

The writer's experience has been entirely in railroad work, and in the remarks that follow he has confined himself to that point of view.

The valuation of common carriers, as outlined by the Federal law, gives a suggestion of "the cost of reproduction less depreciation" and the question immediately arises: "What shall this depreciation be, and how shall it be disposed of?" The writer cannot agree with Mr. Alvord that depreciation is the same for all purposes of valuation, and will attempt to outline the reasons.

Depreciation for Rate-Making and Fair Return.—W. J. Wilgus, M. Am. Soc. C. E., in his paper, "Physical Valuation of Railroads,"* stated the question of depreciation for the purpose of rate-making very clearly and convincingly from the standpoint of the accountant working with an engineer on this subject, and the writer agrees absolutely with those views. The writer believes, however, that there are still other and equally important reasons why depreciation, as the word is commonly taken, should not be deducted from the cost of reproduction new, as of the date of the valuation. Stripped of all theories and technicalities, valuation for the purpose of rate-making becomes a question of allowable rates for a given service due to the corporation from its patrons. The Federal law suggests that these rates shall be determined on a basis of fair return based on the cost of reproduction

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 208.

new less depreciation, but it does not state how this subject of depreciation shall be considered, and has left unsettled a question which is open to much discussion and controversy. The object of the law was to furnish a means of settling many of the disputes that have arisen over the question of rates in the past, and to prevent and settle all such controversies in the future. Such being the case, the consideration of depreciation for rate purposes becomes one of service, and not physical value, unless the service of a given property has fallen below that which might reasonably be expected from a given investment. If it is found, in the valuation of a given property, that the service is less than might reasonably be expected from a given investment, then the property should be assessed a depreciation charge equal to the amount of depreciation from this source. This applies to a well-managed, going property, for if a property has been poorly managed, and there is evidence of needless expense and unwarranted waste, no valuation based on the cost of reproduction of its physical parts would ever give a correct value for rate-making or for any other purpose except that of taxation. Many properties will come between the extremes, but the principle remains the same. Mr. Stone.

Depreciation for Taxation.—The law, as practised, in regard to the taxation of all property, recognizes but one value: the amount that a given property will bring at forced sale; and, for taxation purposes, depreciation assumes an entirely different meaning from that for rate-making; in that it becomes a recognizable physical value, it becomes a problem of actual worth as shown from the inventoried cost new less any depreciation value due from any cause that affects its physical condition. It is a problem of actual physical worth, and cannot include anything but its true dollars-and-cents value. "A" owns a certain lot of land for which he paid $500 and can sell it for this amount at any time that he chooses, but, by waiting a little, he can sell it for $1 000. The $500 is the quick-sale or taxable value, and the larger figure is the sale value. This principle of taxation has been generally recognized by the Courts in taxation cases. The general principle of this subject is very simple, and the writer fails to find any reason for complicating it by the adoption of assumptions that do not exist in every-day practical railroad management; nor does he believe that any valuation based on theoretical assumptions will stand in the Courts. Depreciation for taxation can be the loss due to physical age of a property and nothing else, under the present construction of the tax laws.

Depreciation as Applied to Sales Value.—This phase of the subject is very closely allied to that of taxation, but has many features that prohibit the combination of the two under one head; it is also closely allied to depreciation for the purpose of rate-making, in that it has a value which is dependent on service. There are so many phases to

Mr. Stone. this portion of the subject that the writer does not believe that, at this time, it would be either good business or good engineering to attempt to lay down a general law on the subject; nor, at the present stage of the matter, could a satisfactory general law be laid down. Every carrier has a different problem to solve when it comes to the placing of a sale value on its property. The appraiser has the simplest portion of his work complete when he has finished the physical inventory of the property and made the proper deductions due to the physical depreciation of the property as of any certain date. He has many questions to ask himself: Is the location as it stands the best to be found, considering the traffic? In case the line were sold, would the purchaser rebuild any portion of it for economic reasons? Are there any competing lines built on a more economic basis which affect the earning power of the road under consideration? All these, and many others of a like nature, must be answered equitably before depreciation for sale value can be determined. Such questions as these have values of depreciation, and the appraiser must determine the true values and consider them a part of his appraisal. One road may not have cost double what its rival did, and yet the service of both roads is the same. There is a definite depreciation value here, chargeable against the more costly road, in case an appraiser was placing a sale value on it; but, another critic will say, there would be an appreciation value in favor of the cheaper road in case a sales value was to be placed on it. This is very true; there is a question of which value to apply, but that is for the appraiser to settle. The main point that the writer wishes to make is that there are values of depreciation for sales purposes which do not apply to either the taxation or the rate question.

Mr. Alvord. JOHN W. ALVORD, M. AM. SOC. C. E. (by letter).—It is a matter of great interest that so many keen minds have devoted their close attention to the discussion of this paper. The subject is a rather abstruse one, but it is one that will have to be fully understood by engineers interested in appraisal work. It is, of course, impossible to review the many interesting points brought out in the discussion; only some general thoughts can be here set forth, suggested by the reasoning in certain typical discussions.

The writer is interested to note, from Mr. Burns' discussion, the fact that the California, Idaho, and Missouri Utility Commission Laws require that sinking funds shall be established. To this information may be added the fact that the Indiana Utility Law, not only requires that such funds be established, but permits the reinvestment of such funds in extensions to the property, providing, however, that the value of such extensions shall not be added to the value of the plant as a basis of fair return or purchase.

Since this paper was written, the Society's Special Committee to Formulate Principles and Methods for the Valuation of Railroad Property and Other Public Utilities, has reported, and among its findings there is one (No. 14) in favor of deducting depreciation for the purpose of establishing fair rates. Later, the Committee issued an addition to the Progress Report, in which it amplifies methods of writing off depreciation, and concludes "that any discussion of the subject which does not recognize the inter-relation between the method of providing for the depreciation and the method of valuing the property must necessarily be defective". Attention may be further directed to the fact that, since the paper was written, the Supreme Court, Appellate Division, First Department of the State of New York, in the Kings County Lighting Company *vs.* the Public Service Commission of the State of New York, First District, has upheld the Public Service Commission in its deduction of depreciation from reproduction-cost-new, reasoning out the matter in an opinion of several pages. Mr. Alvord.

The writer is of the opinion, after a careful reading of the discussion and the Progress Report of the Committee on Valuation of December 1st, 1913, and its additional report, that what is needed at this time is a paper on fundamental principles of public utility valuation, and not papers dealing with details of rate-making and depreciation. The confusion of thought existing at this time appears to be caused by lack of understanding of the fundamental principles of utility valuation by a large number of engineers who are now interested in the question, and it will be hopeless to discuss details intelligently until a substantial agreement on such principles is had. As the fundamental principles of valuation work rest in the law and its interpretation by the Courts, many engineers apparently are not generally familiar with them. A brief effort will be made in this closure to clear up one or two of the more important ones which it is necessary to comprehend in order to read the discussions of this paper intelligently.

The writer would select for review, as typical, two of the discussions, one of which has apparently assented to the propositions contained in his paper, and the other of which has dissented therefrom. The first, by Mr. F. Lavis, points out (somewhat diffidently) what seems to him to be the difficulty with that class of valuers who dissent from the deduction of the depreciation from cost new.

To the writer's mind, Mr. Lavis strikes at the root of the matter. The difficulty clearly lies in the fact that the objectors do not take note of the fact that the appreciations of plants and property are included by the reproduction method, and must be considered in connection with the depreciations, and that, if the objectors would exclude depreciation, they must logically exclude the appreciation that the re-

Mr. Alvord. production method usually introduces. The same arguments which one would use to exclude depreciation can be profitably used to show that questions of appreciation should not be entertained. This is fully stated in the paper.

It is a fair guess to say that failure to recognize this fundamental principle arises because some of the engineers now studying these questions for the first time have not yet had the opportunity to serve on actual appraisals, or, if so, they have not served on both sides of the valuation problem, which is quite as important. This appears, to the writer at least, to explain the attitude of some of the more strenuous objectors to the theory here set forth.

No one who has been in valuation work any length of time, and has served on both sides of the controversy, can fail to notice that there are three kinds of mental attitude among valuers:

1st. There are those who approach the subject from the standpoint of the protection of the rights of property and its conservation.

2d. There are those who approach the subject from the standpoint of the rights of the public and their interest in property devoted to public use.

3d. There are those, often recruited from the other two, who are influenced by the sincere desire to find the just and equitable relation between the public interest, on the one hand, and the private interest, on the other.

Now, in these matters we are, at the present time, really studying the third type of problem; that is, the just and proper relation between the public, on the one hand, and property, on the other, and not the protection of capital alone or the public alone.

We have found in the past, and undoubtedly shall continue to find in the future, that the public, in contested cases of valuation, is extremely keen, through its counsel and engineers, to ascertain and deduct every possible item of depreciation that can be taken away from a public utility property, and, on the other hand, that investors and owners, likewise through their counsel and engineers, are fully as keen to observe every fact and element which has appreciated their property. Both these two opposing tendencies are fundamentally sound, and, as usual, the truth lies in a careful conclusion which would allow to both all reasonable depreciation and obvious appreciation. When once the principle is grasped that both depreciation and appreciation must be taken into account, much of the trouble in dealing with this depreciation problem will have vanished. The Constitution of the United States gives the owner of a public utility property the right to have it valued "as of to-day," and as a "property", with all its accretions and growth in value, much of which may not be represented by any actual cash cost; and this right goes along with the right of the public not to pay a return on the full original cost of the property if it can

be shown to have lessened in value in any way. This was stated clearly in the paper, but seems to have escaped the attention of some of those who have discussed the matter. Mr. Alvord.

In the case of railroad properties, for instance, the appreciation of land values must certainly be a very large item, far outweighing any loss in value through minor depreciations of rails, cars, rolling stock, and the like. In the case of electric lighting properties, on the other hand, depreciations may often largely outweigh appreciations.

The second discussion which the writer has selected as typical is that by Mr. Stuart K. Knox. His ideas have been set forth with great clearness and definiteness, and their amiable reasoning is so different from the unpersuasive tone adopted by Mr. Humphreys that one must be tempted, if possible, in the same spirit in which they are made, to note why one must differ from them.

Mr. Knox dissents from the writer's view on the second proposition, and introduces an interesting illustration (in amplification of the general statement in the paper), through which he seeks to show the absurdity of basing rates on depreciated property. He describes a water plant consisting solely of a pipe line, 10 miles long, costing $1 000 000, and having a life of 50 years, through which water, purchased at wholesale, is conveyed and sold at wholesale. A sinking fund of $6 550 is placed in the bank each year at 4% compound interest, which will amortize the entire line at the end of the assumed life. Mr. Knox reasons, as a first proposition, that if the sinking fund was kept intact as part of the property, the rates for service should be predicated on cost new, or, in other words, the accrued amount in the sinking fund should be permitted to earn at the same return as the rest of the property, and he concludes that this proposition, as set forth in the paper, appears to be "unassailable." The second proposition, however, he dissents from; that is, that if depreciation funds are detached or withdrawn from the property and used by the owner elsewhere in private gain, he cannot hope, as a matter of proper protection to the public, to receive rates which include a reserve fund which is not actually in hand, and so in hand that it is really part of the property. Mr. Knox proceeds to show that this proposition is not sound, by supposing that the owner of the pipe line does withdraw his sinking fund allowance and invests it elsewhere outside of the water property at the same rates of interest as it should earn as a sinking fund, and, further, supposing that the owner, in the tenth year of operation, becoming alarmed at the dwindling rates on his water transportation business, restores the fund, and christens it a "sinking fund", as part of the property.

This illustration is typical of a considerable number that have been introduced both in this discussion and in the report of the Committee on Valuation.

Mr. Alvord.

The difficulty of utilizing this kind of illustration is that, not only is it quite unlike anything that happens in practically operated utilities, as has been set forth at length in the paper, but the further and more fundamental difficulty that it is not applicable to the problem here presented, because it is, pure and simple, the amortization of an original and actual cost. In valuation work, we are not considering the retracing of a past investment and its exact mathematical protection, at all, but we are considering present-day valuations made on a live and growing property in its entirety.

So far as the writer's knowledge goes, it is not customary to treat the actual past cost as something from which depreciation should be deducted. It is true that the representatives of the public often make a plea for such a procedure, but Courts, commissions, or appraisal boards, with which the writer is familiar, have not decided that depreciation should be deducted from actual past cost for the purpose of making rates unless it can be shown that past cost is present value, nor does this paper even intimate the possibility of such a proposition.

What has been attempted to set forth is that the reproduction method—that is, cost new as of to-day—if used for determining values, should have deducted from it the depreciation before using it for the computation of rates, if an adequate fund for depreciation is not in hand. A careful re-reading of the paper will make this clear. It is true that Mr. Grunsky calls it "past investment", but the writer only quotes Mr. Grunsky's exact language to make his own distinction, that it is "property now" that is valued, more conspicuous.

Now, Mr. Knox, in all his discussion, is evidently dealing with past cost, because in one place (page 850) he says: "Assume that the reproduction-cost-new of the pipe is $1 000 000", and in the very next paragraph he says: "In the construction of this pipe line, the owner converted $1 000 000 of money into physical plant."

"Reproduction-cost-new" with him evidently means original cost, so far as his illustration and argument is concerned at least, for that idea runs through all of his succeeding reasoning on this illustration. On page 857 he says:

"On the other hand, if we neglect the fluctuations in the prices of labor and materials, reproduction-cost-new will remain an unvarying quantity as long as no additions are made to the property."

And, further:

"The reproduction-cost-new having once been ascertained may subsequently be kept up to date merely by adding the cost of extensions as these are made, and the 'adequate' annual depreciation reserve."

The writer has been actively engaged for the past 15 years in valuation work, and has never found yet that reproduction-cost-new would remain an unvarying quantity, even in plants which were not being

extended. Nor can he understand how the value of a property, having once been ascertained, can subsequently be kept up by adding the cost of the extensions, for properties usually grow in value or decrease in value with the growth or decrease of the surrounding population and the general increase or decrease in opportunity to give service, and this entirely outside of and irrespective of the money actually put into them. Mr. Alvord.

Mr. Knox must unquestionably realize, as any engineer would realize on second thought, that utility properties in actual life may appreciate or depreciate in value outside of their original cost, and that, if growth is found, it is conceded to be the property of the owner, by the Constitution of the United States, as interpreted by the Supreme Court in numerous cases.

Mr. Knox's discussion, therefore, has only been taken as typical of this neglect to take into account possible appreciation, but he is followed in this respect by no less than six out of the sixteen discussions. The writer will try to make this fundamental difference clear.

"Past cost" is commonly understood to indicate the actual monies originally expended for the upbuilding of a property. It is one of the elements to consider in reasoning from cost evidence to value, but the Courts generally hold that it is not a controlling element.

Past cost is seldom a good measure of value except in the case of quite recent expenditures. Past cost may have in it monies expended for obsolescent structures, but it contains none of the appreciations which commonly accrue to a property, and often accrue without actual expenditure. Because it has none of these appreciations, it is not proper to deduct from past cost the appreciations; and in actual practice it is not done unless some attempt is made to compute and add the appreciations as well.

Reproduction cost is the method of computing the cost to reproduce the "property" as of to-day, in a manner that is humanly possible, and at market prices for labor and material, and including the development of the business. Consequently, it contains many of the appreciations that have come to the property by reason of the growth of its environment. Not all of these appreciations have cost actual cash; some of them are natural accretions or earned and unearned increments, which have cost the owner nothing. From such an estimate should be taken the depreciations which the existing property has suffered by reason of its age, changing conditions, or use. After this is done, if the result, in connection with other lines of evidence, is found to be property value, as distinguished from cost, then that value may be taken to estimate the fair return.

Now, all this is elementary, and in preparing the paper it was assumed that these fundamental principles were understood, but to guard fully against misconception at the very outset, the paper states that

Mr. Alvord. we are discussing only that line of evidence as to value known as the reproduction method, or cost-new-less-depreciation.

Mr. Wilgus speaks of a balance sheet showing an excess of assets over liabilities. This surely relates to past cost. Farther along he speaks of an "impaired investment", showing that he has investment in mind and not reproduction-cost-new.

Mr. Willoughby says: "it is the ultimate finding of the accounting * * * that fixes the value of the utility," and though he immediately apprehends that we are limited to reproduction methods, yet he later speaks of such methods as though they were recent past cost, and in his illustration lapses into past cost completely again.

Mr. Boes clearly does not have in mind that we are discussing a "property", the value of which is fixed as of to-day.

Mr. Vensano argues entirely from past cost, and uses past cost in his illustration.

The Special Committee on Valuation of Public Utilities argues entirely from original cost, and, in its illustrations, appears to be constantly thinking about the proper protection of original cost.

Now, taking Mr. Knox's example, and conceding his assumptions; that is, that original cost is "value" continuously through the life of the plant, the writer would promptly concede the correctness of his argument, that depreciation should not be deducted in any case for the purpose of making rates. There can be no rational dissent from such a proposition.

Practically, however, past cost is almost never present value. A thing may be worth what it cost, or it may be worth much more, or it may be worth much less; hence, past cost is very rarely accepted by the Courts as the best evidence of present value.

Even reproduction cost, less depreciation, is not always value. A plant may often be worth more or may be worth less than it would cost to reproduce it, and though some utility properties depreciate more rapidly than they appreciate, as a usual thing they appreciate much more rapidly than they depreciate, and both facts must be taken into account. In other words, original cost, plus the algebraic sum of all the changes in cost and theoretical depreciations, should equal reproduction cost, less depreciation, as of to-day, if it were humanly possible to make such an analysis correctly, but even so, we will not have arrived at value; other matters have to be considered as well.

In fixing fair rates, the Courts have said, over and over again, that it is the fair value (not cost) of the property used and useful for the public that must be taken into account.

It is interesting to note that most of the water-works engineers who have had experience in appraisal work have not misapprehended the fundamental proposition here set forth. Mr. Kiersted and Mr.

Burns, particularly, have thrown a great many interesting side-lights on the main question. Mr. Alvord.

To emphasize the matter further, it may not be inappropriate to revert to Mr. Humphreys' illustration of the hen (page 861 of Mr. Knox's discussion). Like a good many illustrations that do not illustrate, this one is misleading, in that it is not comparable with the actual conditions as we find them, for, to make the illustration comparable with ordinary utility problems, we should imagine that the hen lays more eggs as time goes by than she did originally, and is at the same time growing old.

What is the value of this hen to the consumer? Clearly more than it originally was, modified by the fact that she might soon die, and whatever the philosophy of the hennery farm manager, the Courts and commissions and practical appraisers would take both facts into account in fixing the price of eggs, especially if there were only one hen.

In conclusion, the writer is of the opinion that the trend of public utility regulation in the future is going to lean more and more to the requirement, in municipal utilities at least, that funds for depreciation, renewal, and contingency shall be largely, if not entirely, kept in hand as part of the property, subject to the control and supervision of the commissions. Such funds are not necessarily idle, as has been suggested, nor are they withdrawn from useful activity, but the public is undoubtedly much better protected, and the property is stronger and the owners' credit better when such funds are within public control and supervision.

It is a significant fact that in several cases, which have come under notice of late years, where new financing of public utilities, not under commission control, has been required, that the bankers have actually insisted on a replacement fund being created, and constantly maintained to a required amount, in order to make the property more sound and the securities issued more secure.

The case of the steam railroads presents a somewhat different aspect from the ordinary municipal utility. So much of the property of the steam roads is in land, which does not usually depreciate, and the remaining property depreciates so rapidly that, as pointed out in the Third Avenue Railroad case by Mr. Floy, it may be difficult and undesirable to create a special fund for depreciation, or find any method that would be more simple than that of maintenance, pure and simple. This, however, should not preclude the deduction of depreciation from reproduction-cost-new in cases of valuation where appreciations had been duly considered.

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Paper No. 1298

A STUDY OF FLUID RESISTANCE.

By Luther Wagoner, M. Am. Soc. C. E.

The study of hydrodynamics by the usual method of differential equations, despite the fact that the brightest mathematical minds have attempted the subject, has thus far brought forth no results of a practical nature applicable to such fluid resistances as the steady flow in pipes, or for the resistance of a fluid at rest to the steady motion of a solid in the fluid.

The subject is full of difficulties and nearly impossible of any direct treatment by known mathematical methods.

For a long time the writer has held the belief that there is such a similarity in the nature of the resistances of a solid moving in a fluid, or of a fluid moving in a pipe or channel, that it might be profitable to make comparison of the motions of each, using such experimental data as are available, and that such a joint study might also prove useful in that any information thus gained about one would be helpful to resolve any doubts about the other. The general method used is that of a logarithmic diagram of experimental results obtained, combined with simple mathematics sufficient to explain the locus of the lines thus found. The results obtained represent a considerable advance in information about both subjects. Summarizing what follows, it is shown that, for bodies moving in a fluid at rest, or for a

fluid moving along a surface at rest, there is for both the same general law of resistance; and, also, as a result of such studies, there is presented to the hydraulic engineer the first rational theory of the laws governing the flow in pipes.

There being but scanty information about the motion of bodies in a fluid, this study was made first, and from the results found the way was made easier for the study of fluid motion in pipes, and they are here given in the order named.

PART I.—THE TERMINAL VELOCITIES OF BODIES IN A FLUID MEDIUM.

A body falling, or rising, in a fluid medium does so with accelerated velocity and increased resistance; when the resistance becomes as great as the impelling force, there is uniform motion, more commonly called the terminal velocity. (In a frictionless liquid, this would not occur, but in an actual liquid it does occur, and uniform motion is attained very rapidly.)

The Experimental Data.—For the purposes of this paper, the writer has selected data from the following sources:

"Velocity of Galena and Quartz Falling in Water," by Professor Robert H. Richards.* These are grains of mineral from 11.93 to 0.00152 mm. in diameter, but have a special value due to the long range covered.

Mr. H. S. Allen.† Steel balls in water, amber spheres in water, paraffin wax spheres rising in aniline, air bubbles rising in water and aniline.

"On the Viscosity of Liquids," by Mr. O. S. Jones.‡ The determination of the viscosity of glycerine by dropping small spheres of mercury and noting the diameter and velocity.

"On the Maximum Velocity Acquired by Small Bodies Falling in Water and Glycerine," by the writer.§

Also, for confirmatory use, "The Terminal Velocity of Small Spheres in Air," by Messrs. John Zeleny and L. W. McKeehan.||

* *Transactions*, Am. Inst. of Min. Engrs., 1907, pp. 210-235.

† *Philosophical Magazine*, Vol. 50, 1900.

‡ *Philosophical Magazine*, 1894.

§ *Transactions*, Technical Soc. of the Pacific Coast, Vol. V, p. 32, 1888.

|| *Physical Review*, Vol. CLXIX, p. 535, May, 1910.

Notation.—The centimeter-gramme-second system of units is used.

D = Diameter of the grain or sphere;

s = Specific gravity of the solid;

Δ = Difference of specific gravity of the solid and the fluid medium;

V = Terminal velocity of the body;

D_c = Critical diameter;

V_c = Critical velocity;

μ = Coefficient of viscosity = in water to

$$\frac{0.017944}{(1 + 0.023121\,T)^{1.5423}},$$

ν = Kinematic coefficient of viscosity $= \frac{\mu}{\rho}$;

ρ = Density of the fluid medium;

K_0, K_1, K_n, K_2 = Coefficients;

$a, b, c,$ = Constants or general coefficients;

θ = Coefficient of form, or shape of body;

ϕ = Coefficient of roughness, or surface condition;

n = Exponent;

log. = Common logarithm;

tan. = Tangent;

tanh. = Hyperbolic tangent.

TABLE 1.—Physical Constants of the Data.

Material.	Form.	Specific Gravity. S	Medium.	Density. ρ	Δ.	μ.	Experimenter. Remarks.
Galena........	Grains...	7.50	Water	1.00	....	0.01 ±	Richards.
Quartz	Grains...	2.65	Water	1.00	....	0.01 ±	"
Steel	Balls.....	7.731	Water	1.00	6.731	0.0125	Allen.
Amber	Spheres..	1.07683	Water	1.00	0.07683	0.011	"
Paraffin	Spheres..	0.907	Aniline	1.038	0.131	0.06023	"
Mercury........	Spheres..	13.59	Glycerine..	1.260	12.33	10.69	Jones.
Glass........	Spheres .	2.61	Glycerine..	1.24	1.37	3.428	Wagoner. Viscosity computed.
Air	Bubbles .	0.0012	Water.....	0.9988	1.00	0.01404	Allen.
Air	Bubbles .	0.0012	Aniline ...	1.0368	1.038	0.06023	Allen.
Lycopodium..	Spheres..	1.175	Air	....	....	0.000185	D=0.00316 V=1.76
Polytrichium	Spheres..	1.53	Air	....	....	0.000185	D=0.000956 V=0.228
Lycoperdon..	Spheres..	1.44	Air	....	....	0.000184	D=0.000418 V=0.0467

Graphical Representation of the Experimental Data.—For the purpose of discovering laws, logarithmic co-ordinates were at first used where log. D was an abscissa, and log. V was an ordinate. This was not

entirely satisfactory, especially for the grains. The unavoidable uncertainties about the diameter and velocity give (as can be seen by reference to the original paper) a certain impression of indefiniteness as to what actually happened; or, if any sharply-defined critical points exist, this, coupled with the smooth free-hand curve drawn through the points, misled the writer for some time as to the true nature of the data. Although it is a subject for regret that information was not given as to weights of grains, surface, and viscosity, the original paper, because of its long range, is still very valuable, and without its aid the writer would not have attempted this paper.

In casting about for new devices for logarithm platting, an original method, and believed to be new, has been used.

Theorem.—If $V = f\,D^n$ and both V and D are affected by probable errors of $\pm\, p$ and $\pm\, q$, respectively, then $V\,D^{\pm 1} = f\,D^{n \pm 1}$ is the most probable value of the new function.

Proof.—This is proved by the direct multiplication of $V \pm p$ by $D \pm q$, when both p and q disappear, or graphically by platting four possible points of $\frac{V}{D}$ and taking the center of gravity of the figure as the most probable point.

Plotting log. $V\,D$ as ordinate over log. D as abscissa gives a longer and smoother curve, but the angles of intersection of the different branches of the curves are not as well defined as by using log. $\frac{V}{D}$ as ordinate and log. D as abscissa, and this method has been used and is shown on Plate XVI, where it is seen that, for the grains, a very satisfactory definition of the intersections of the three branches has been obtained; also—which is new—it is seen that, for the first time, there is shown to exist for all bodies and fluids three stages of flow, their exponents being 1, *n*, and 2; and each is accompanied by a rather sudden transition from one stage to another (which might be compared to the action of a balance; when equilibrium is reached there is unsteady motion and when passed the beam drops).

Explanation of Plate XVI.—It is observed that for the first stage the slope of the lines for $n = 1$ is tan. $= 1$, and for $n = 2$, the slope is tan. $= -\frac{1}{2}$; from which there is found

$$n = \frac{3}{\text{tan.} + 2} \quad \text{.......................} (1)$$

If log. V had been platted over log. D, then the relation would have been

$$n = \frac{3}{\tan. + 1} \quad \cdots\cdots (2)$$

and

$$\tan. = \frac{3}{n} - 1 \quad \cdots\cdots (3)$$

Plate XVI has been used for the purpose of finding n and the tangent, and the coefficients, K_1, K_n, K_2, etc.; the results are tabulated as if the diagram was in accordance with Equations (2) and (3), (or log. V as ordinate).

The effect of using log. $\frac{V}{D}$ as ordinate, is to increase greatly the angle of intersection of the first and second branches of the curves. Thus, for galena, where $n = 1.82$, and using each form of ordinate, there is for each branch the following angles:

	$n = 1$	$n = 1.82$	$n = 2$	
Ordinate $= V$	$+ 63° 26'$	$+ 33° 45'$	$+ 26° 34'$	differences $= 17° 41'$, $7° 11'$
Ordinate $= \frac{V}{D}$	$+ 45°$	$- 18° 20'$	$- 26° 34'$	" $= 63° 20'$, $8° 14'$

This great increase of intersection angle, where n is greater than 1.50, makes the locus of the lower critical point more easily found from the data; making the ordinate $= VD$, gives a longer and smoother curve with greatly reduced angles of intersection, and in certain cases all three methods might be found helpful if there is much irregularity of data.

Table 2 shows the data contained on Plate XVI.

TABLE 2.—DATA TAKEN FROM PLATE XVI.

Material.	Medium.	log. D_c	log. V_c	log. K_1	log. K_n	log. K_2	tan.†	Exponent n.
Steel balls	Water	....	....	....	....	2.2043	....	2.00
Galena grains	Water	8.02*	0.74	4.700	2.013	1.935	0.6487	1.82
Quartz grains	Water	8.262	0.37	3.846	1.723	1.546	0.7928	1.673
Air bubbles	Water	8.1837	9.9659	3.5985	2.3880	....	1.3335	1.2857
Air bubbles	Aniline	8.426	9.848	2.996	....	....	1.558	1.173
Amber spheres	Water	9.086	0.363±	2.1907	1.2646	....	0.9865	1.510
Paraffin spheres	Aniline	8.993	9.9647	1.9787	1.3566	....	1.3823	1.2592
Mercury spheres	Glycerine	....	....	1.7984	....	....	....	....
Glass spheres	Glycerine	9.1166	9.5602	1.3270	....	....	1.444	1.2275
Glass spheres	Water	8.3226	0.5263	3.8823	1.7665	....	0.739	1.7250
Lycopodium spheres	Air	....	....	5.246	....	....	....	1.0
Polytrichium spheres	Air	....	....	5.359	....	....	....	1.0
Lycoperdon spheres	Air	....	....	5.427	....	....	....	1.0

* = 8.02 — 10.

† The tan. is scaled from the plat, and 1 has been added to it to make it of the form, log. V = ordinate.

PLATE XVI.
TRANS. AM. SOC. CIV. ENGRS.
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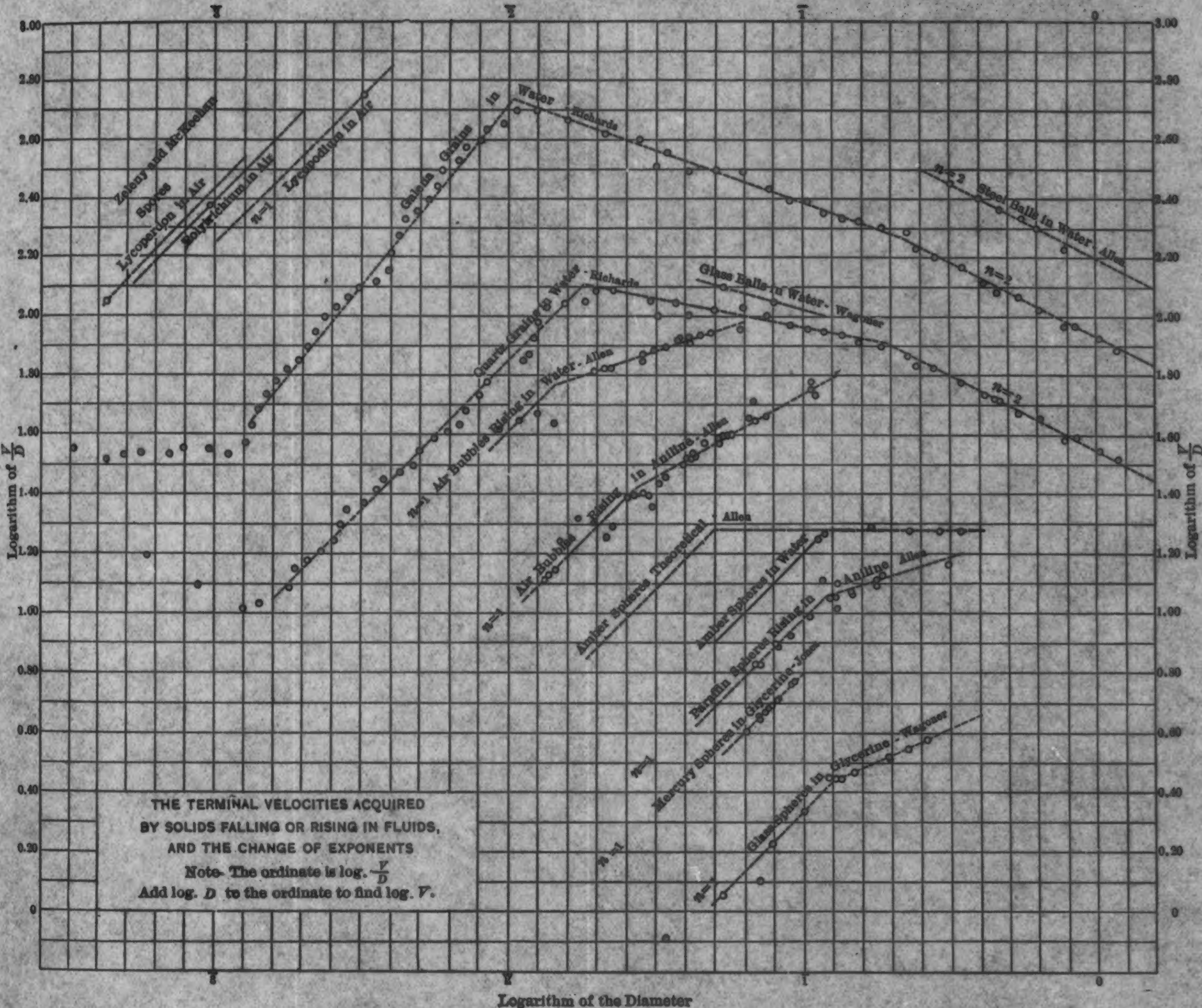
WAGONER ON
FLUID RESISTANCES.



THE TERMINAL VELOCITIES ACQUIRED BY SOLIDS FALLING OR RISING IN FLUIDS, AND THE CHANGE OF EXPONENTS

Note- The ordinate is log. $\frac{V}{D}$

Add log. D to the ordinate to find log. V.

From the data in Table 2, in which there are many gaps, it will be attempted to find the missing parts, and the result of the investigation appears as dotted or full lines on Plate XVI.

On the Present State of Knowledge Concerning the Terminal Velocity of Bodies, Falling or Rising, in Fluids.—An immense amount of purely analytical work has been done by various eminent mathematicians and physicists in an effort to solve the problem, both for viscous and non-viscous fluids, but with few notable results of an applicable nature, probably for the reason that the theoretical work did not follow and interpret experiments, rather than precede them.

Sir Gabriel Stokes, many years ago, gave a solution for very small bodies falling in a viscous medium. The solution is correct, but is limited in its application by a condition, namely, radius^2 $\times$ velocity $\div$ viscosity, must be small; that is to say, the higher powers of $\dfrac{r^2\,V}{\mu}$ can be ignored. In fact, Stokes says that the solution was obtained by neglecting the higher-power terms of the general equation. Concerning the higher velocities, it has long been known that the resistance varies as V^2 and is independent of the viscosity. For still higher velocities, such as for cannon balls, the resistance varies as a power intermediate between the second and third. About the powers intermediate between 1 and 2, nothing is known, nor is the state recognized by any previous writers. It thus appears that if the diameter be uniformly increasing, the order of exponents will be:

D_1 to D_2	—Resistance	varies	as	V^1
D_2 to D_3	"	"	"	V^n
D_3 to D_4	"	"	"	V^2
D_4 to D_5	"	"	"	V^{2+e}

and the transition from an exponent to the next higher will be somewhat abrupt. (This is subject to later remarks about a possible change of exponent for D_0 to D_1 where $R \backsim V^{1+}$).

Working Hypothesis.—Mr. H. S. Allen, whose observations are used in this paper, proposed a general formula to cover all values of the exponent. He assumes a resistance

$$R = k\ \text{Velocity}^x\ \text{Radius}^y\ \text{Viscosity}^w\ \text{Density}^z;$$

and, using dimensional methods, the above becomes

$$\text{Force} = \text{Resistance}.$$

$$M\ L\ T^{-2} = k \left\{ (L^x\,T^{-x})\,(L^y)\,(M^w\,L^{-w}\,T^{-w})\,(M^z\,L^{-3z}) \right\} . (4)$$

$$\begin{array}{llll} \text{From } M & \text{there is} & 1 = & w + z \\ \text{“} \quad L & \text{“} \quad \text{“} & 1 = & x + y - w - 3z \\ \text{“} \quad T^{-2} & \text{“} \quad \text{“} & -2 = & -x - w \end{array}$$

This can be solved by assuming $x = y = n$, when the resistance becomes

$$\text{Resistance} = k\,(V\,D)^n\,\mu^{2-n}\,\rho^{n-1} \ldots\ldots\ldots\ldots (5)$$

For spheres, using diameter instead of radius, the completed equation is:

$$D^3 \frac{\pi}{6}\,\varDelta = k\,(V\,D)^n\,\mu^{2-n}\,\rho^{n-1} \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Equation (6) is perfectly general and is correct, provided the fundamental assumption is correct. In its present form, it must be considered as quite elemental, as the coefficient, k, is not provided for; as will be shown later, it is a complex quantity, and additional terms, of the form, ϕ^{n-1}, and z^{n-1}, are required to express the resistance completely.

Apparently, Mr. Allen contented himself with proposing the formula, but does not appear to have developed it further, or put it to any experimental test. There appears to be a considerable degree of probability that Equation (6) can be made to represent experimental data accurately, and it will be used as the basis of what follows.

Application of Equation (6), for $n = 1$.—Making $n = 1$, Equation 6 reduces to (for spherical bodies)

$$D^3 \frac{\pi}{6}\,\varDelta = \frac{C\,V\,D\,\pi\,\mu}{g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

where C is a constant, probably related to the form of the body. π is introduced on the resistance side as a surface factor of D, and g is the constant of gravity. For cases where $\dfrac{D^2\,V}{\mu}$ is small, then C is exactly 3; hence, for normal spheres,

$$g\,D^2\,\varDelta = 18\,V\,\mu \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

and lastly, making $D = 1$, there is

$$V = k_1 = \frac{g\,\varDelta}{18\,\mu} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

where

$$K_1\,D^2 = V \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

Equation (8) gives the locus of the lines quite accurately for air bubbles, mercury, and glass bodies. For the light bodies (amber, $\varDelta = 0.07683$, and paraffin, $\varDelta = 0.131$) the foregoing value of 18 of Equation

(8) no longer holds true, and a correction for inertia is required. As a practical method, it is suggested to add to 18, $\frac{144}{g\,\Delta}$ for falling bodies and $\frac{72}{g\,\Delta^2}$ for rising bodies.

Using Equation (7), these corrections would be added to $3\,\pi$. For the case of irregular bodies, and generally hereafter, the quantity, $3\,\pi$, or its equivalent, will be called θ, such that, calling w a factor for the effective weight-producing motion, there is

$$D^3\,w = \frac{\theta\,D\,V\,\mu}{g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

General Consideration of Equation (6).—Equations (7) to (11) show that the coefficient, k, of Equation (6) vanishes for $n = 1$, also that the quantity, θ, may be considered a factor related to the form of the body and also of its inertia terms. (It is probably the velocity function of hydrodynamical writers.) As it enters as a constant for any value of μ, it may be considered as joined to $V\,D$, or the resistance is of the form

$$\frac{1}{n}\left(\frac{V\,D\,\theta}{g}\right)^n.$$

If a friction coefficient is assumed to exist, and it seems probable that the roughness of the surface must affect the motion in the stages where n is greater than 1, then it must be of the form, $\phi^{(n-1)}$. Also, a stream function can be considered a factor with similar exponent. Calling z the product of a fluid density, friction coefficient, and a stream function, then the final expression becomes

$$k\,(n - 1)\,z^{n-1}$$

the term, $n - 1$, being required to cause the whole expression to vanish, or become 1, when $n = 1$ Equation (6) then becomes, equating weight to resistance:

$$\text{Effective weight} = \frac{1}{n}\left(\frac{V\,D\,\theta}{g}\right)^n \mu^{2-n}\,k\,(n - 1)\,z^{n-1} \ldots . (12)$$

Equation (12) can also be written

$$\text{Effective weight} = \frac{\mu^2}{n\,z}\left(\frac{V\,D\,\theta\,z}{g\,\mu}\right)^n k\,(n - 1) \ldots\ldots\ldots\ldots (13)$$

in which form it is seen that the terms, $\mu^{(2-n)}$ and $z^{(n-1)}$, of Equation (12) do not require integration coefficients other than the $\frac{(n-1)}{n}$ common to both equations.

Equating resistances of Equations (11) and (12), for the common V_c point, and reducing, there is

$$\mu = \left(k \frac{n-1}{n}\right)^{\frac{1}{n-1}} \frac{V_1 D_1 \theta z}{g} \ldots\ldots\ldots\ldots(13)$$

making $n = 2$ and equating resistances for the common point or upper V_c, and reducing, there is

$$2\left(\frac{n-1}{n}\right)^{\frac{1}{2-n}} \mu = \frac{V_2 D_2 \theta z}{g} \ldots\ldots\ldots\ldots(14)$$

making V_1 D_1 of Equation (13) $= k_1 D^3 = \frac{g \Delta}{\theta \mu} D^3$, Equation (13) becomes

$$\frac{\mu^2}{\Delta z \left(k \frac{n-1}{n}\right)^{\frac{1}{n-1}}} = D_c^3 \ldots\ldots\ldots\ldots(15)$$

Special Value of $n = 2$.—Making $n = 2$, Equation (12) reduces to

$$2g \text{ weight} = kz\,(V D \theta)^2 \ldots\ldots\ldots\ldots(16)$$

which shows that in this stage the motion is independent of the viscosity, the latter having become μ^{2-2}, or 1. For exponents greater than 2, it is seen that μ enters the resistance with a minus exponent, thus increasing the resistance.

Approximate Values of n and D_c.—In a general way, the close of the $n = 1$ stage, or V_1 D_1 tends to a constant value, or

$$\log. V_1 D_1 = \bar{2}.68 \pm = 0.048.$$

$$\log. K_1 D^3 = \bar{2}.68 \pm = 0.048.$$

This seems to be true for the regular bodies, and subject to the conditions that $\frac{V D^2}{\mu}$ is small.

Calling $\left(\frac{3}{n} - 1\right)$ a tangent, or the actual slope line on a logarithmic diagram of the nth stage, then the following relation is quite close:

$$\tan. = e\, D_c^{\frac{1}{3}} \ldots\ldots\ldots\ldots(17)$$

where e is the Napierian base and D_c is a critical diameter, and from this is found

$$n = \frac{3}{e}(\log. k_1)^{\frac{1}{3}} \ldots\ldots\ldots\ldots(18)$$

Equations (17) and (18) are to be considered as approximations, subject to the above named limitation, that for regular bodies $\frac{V D^2}{\mu}$ must be small. Thus understood, they are useful for planning experiments or checking doubtful results.

Extremely Low Values of D and V_1.—Examining Plate XVI, it is seen that at log. $D =$ galena 3.04, quartz 3.14, an abrupt change occurs, the exponent increasing to 1.50, 1.60 $\pm$. Professor Richards attributed this to convection currents. To the writer this appears to be a change in boundary conditions such that there can be and is an actual change of exponent.

Physical Concept.—Assuming that there is such a change of the law of exponents as just suggested, the whole subject might be viewed as follows:

When V is very small, suppose that a certain quantity of fluid be attracted to the body, such that its radius, R, becomes $R + r$, the line of slip then being over the greater boundary, this would also change the Δ, or density relation, so that for a given weight there would be a greater resistance, and hence an increased exponent. The compound grain or sphere could also be imagined to deform under increasing velocity, passing from spherical through elliptical forms and lastly reaching the condition of a quiet wake, when the $n = 1$ stage begins. At the close of the $n = 1$ stage the wake becomes turbulent, and at the end of the nth stage a new boundary condition begins; also, at the end of $n = 2$ another change in the boundary condition must occur.

Descending the scale of diameters, it seems probable that another law relating to the colloidal state of matter must enter about here as a factor, and the diameters where this change occurs (0.01 to 0.015 mm.) are about those required for the grains of Portland cement to become active (colloidal). Lastly, it is known of most colloids that when the diameter is sufficiently small there is permanent suspension. It would be interesting to test out Portland cement in this way, using a suitable fluid, and see whether a curve such as above noted would be coincident with the limit of size of the active grains as distinguished from the larger but inert grains.

If this view of an increased exponent should prove to be correct, then the order of succession of exponents is

$$\text{Exponents} = (1 + a), (1), (1 + b), (2), (2 + c),$$

which conveys the idea of periodicity of the exponent, or, if not periodic, it suggests the idea that the above changes of exponent are

subject to some law, at present unknown, which offers to become a profitable field for research work.

Conclusion.—It has been shown that bodies falling or rising in a fluid, on reaching their terminal velocities, follow a definite law of exponents as long as the boundary condition is unchanged. At or about such change there is a condition of unstable equilibrium, such that a slight increase in the diameter and velocity is accompanied by a sudden change in the law of resistance. For the further study of the question there are two methods: (*a*), by the general methods of differential equations, to attempt to deduce what law of resistance will be encountered; and (*b*), to make an explanation of the results found by proper mathematical reasoning. The first method gives no check on the result obtained, and the second method begins with a definite result and seeks a satisfactory explanation. The method, as can readily be seen, only requires four experiments for finding the constants, n, k, and z, of Equation (12); one experiment on each side of the lower and upper critical points suffices to give the locus of all the points of the curve. If further experimental and mathematical work shows that the constants, n, z, and k, are computable, then it follows that it must be possible to separate the roughness coefficient, ϕ, of z, and evaluate it, and thus permit of microscopic research on the nature of the surface as related to its coefficient, which would doubtless throw much light on the nature of a roughness as applied to pipes.

Referring to Equations (17) and (18), where the approximate equations for n involve the coefficient, k, which contains μ^{-1}, or n is a function of μ and anticipating the conclusions of Equation (36) of Part II, where n is not a function of viscosity, it may be said that in the case of solids the field is unlimited and the turbulent motion set up gradually dies away by the damping action of viscosity, though, in the case of pipes, the field is limited and there is no such damping action; hence it is reasonable that both conclusions are correct.

Part II.—A General Formula for the Resistance to Flow of Fluids in Pipes or Channels.

The notation used is the same as in Part I, with such additions as are required. The centimeter-gramme-second system of units is used throughout.

Preliminary Considerations. Three Stages of Flow Considered.

First Stage.—Below a certain critical or limiting velocity, the law of flow for any pipe or fluid is expressed very accurately by the equation

$$K_1 D^2 S = V\mu \dots\dots\dots\dots\dots\dots\dots (1)$$

Second Stage.—Between the foregoing limiting velocity and a still greater one, or the lower or higher critical velocities, is a region of flow concerning which but little is known. It has been recognized heretofore by writers and experimenters as a zone, or region, of disturbance. It will be shown that for any pipe there is a quite simple law of flow for this stage (hereinafter called the second stage), and that it is represented accurately by (making the density $\rho =$ one)

$$K_3 S\mu = V^3 \dots\dots\dots\dots\dots\dots\dots (2)$$

Third Stage.—For all velocities above the higher critical velocity (end of the second stage), which is the case of ordinary use, or turbulent flow, there is no theory thus far developed except the empirical relation of

$$C R^{\frac{1}{2}} S^{\frac{1}{2}} = V \dots\dots\dots\dots\dots\dots\dots (3)$$

where the coefficient, C, is a variable. This is the oldest formula, and the efforts of hydraulicians have centered largely about correct values for the coefficient, C.

There is also the exponential type of equation

$$C R^y S^x = V \dots\dots\dots\dots\dots\dots\dots (4)$$

in which C is not constant and the exponents, y and x, are thus far not within the scope of prediction, but do represent the facts quite accurately, as determined experimentally.

Viscosity as a Factor.—Experiments show that the viscosity of the fluid affects the flow, and that in certain cases it is quite a notable factor, and cannot be disregarded; also, that its relation to the velocity is exponential, such that its consideration makes Equation (4) become

$$C R^y S^x = V\mu^z \dots\dots\dots\dots\dots\dots\dots (5)$$

Here C, again, has a new value, and is not a constant.

The Coefficient of Roughness, ϕ.—It has long been known that the surface condition of a pipe, or channel, influences the flow profoundly. Provisionally, this factor will be called ϕ, as in Part I,

leaving open the question of the nature of its experimental value as hereinafter found; that is, whether or not the value as experimentally found is the actual coefficient, or is some function of the actual coefficient.

Making R, μ, and ϕ each equal to 1, then for a given pipe there can be written

$$C\,S = V^n \qquad (6)$$

where C is a constant. Taking all known experimental data, it is seen that an equation of the form of Equation (6) corresponds most nearly to the data.

Logarithmic Co-ordinates.—Making log. S the abscissa, and log. V the ordinate, the exponent, n, of Equation (6) is found from the straight-line relation of

$$\frac{\log. S_1 - \log. S}{\log. V_1 - \log. V} = n \qquad (7)$$

In general, it may be said that the experimental values of the exponent, n, for all the cases where any approach to accuracy has been made, ranges from $\frac{5}{3}$ to $\frac{6}{3}$, and never surpasses 2, in value. For the few cases where n has been found greater than 2, there is usually some abnormality. (This matter will be discussed later.) For flumes and channels, n never reaches the value of 2. To prove this, take the Kutter formula, as probably best expressing the law of flow, and assume only the variables present, such that

$$V = fx^n \qquad (8)$$

and

$$\frac{1}{n} = \frac{V\,dx}{x\,d\,V} \qquad (9)$$

Make the Kutter coefficient of roughness, n, and the slope, s, constant, and let R be the above variable, x; then the formula can be written

$$V = \frac{A\,R^{\frac{1}{2}}}{1 + \dfrac{C}{R^{\frac{1}{2}}}} \qquad (10)$$

Differentiating and reducing to Equation (9), it is found that the exponent, n, is

$$\frac{1}{n} = \frac{\dfrac{1}{2}}{1 + \dfrac{C}{R^{\frac{1}{2}}}} + \frac{\dfrac{C}{R^{\frac{1}{2}}}}{1 + \dfrac{C}{R^{\frac{1}{2}}}} \qquad (11)$$

from which it is seen that $\frac{1}{n}$ varies from 1 to $\frac{1}{2}$, between the limits of $R = 0$ and $R = \infty$, or the resistance varies as R^1 to R^2.

Making the slope, s, the variable in Equation (8), the formula becomes

$$V = \frac{A + \frac{e}{s}}{1 + \frac{C}{s}} \quad \text{.....................(12)}$$

from which, by Equation (9), the exponent is found to be

$$\frac{1}{n} = \frac{\frac{A\,s^{\frac{1}{2}}}{2} - \frac{b}{2\,s^{\frac{1}{2}}}}{A\,s^{\frac{1}{2}} + \frac{b}{s^{\frac{1}{2}}}} + \frac{\frac{e}{s}}{C + \frac{e}{s}} \quad \text{.....................(13)}$$

which also makes the resistance vary as the first power when s is small and approaches the second power of V when s is large. Hence, in general, the Kutter formula can be expressed exponentially with the exponent of V ranging from 1 to 2.

From the foregoing it can safely be said that, for flow in the third stage, the resistance varies as V^n, with values of n ranging from 1 to 2, or, more closely, as far as is known for pipes conveying water, n ranges from $\frac{5}{3}$ to $\frac{6}{3}$ or more simply.

The third stage is characterized by the presence of a variable exponent, n, with $n > 1$ and $n < 2$.

Comparing the foregoing, with the deductions of Part I, there is seen to be the following order of succession of exponents:

	First stage exponent.	Second stage exponent.	Third stage exponent.
Solids	1	n	2
Pipes	1	3	n

As far as can be judged from the experimental data (which are far from complete), the transition from one stage to another is quite sudden or abrupt; or a very small increase in the driving force causes the foregoing changes in the exponents of the law of resistance.

Distinction of Boundary Conditions.—For the solid moving in an unlimited field of fluid, the maximum stream line disturbance is at the surface of the moving solid; and, due to viscosity, there is a damping

action which quickly reduces the disturbance to zero at no great distance normal to the axis of motion. For pipes, where the second or third stages are considered, there is quite a different law. The fluid friction created by the boundary conditions causes turbulent motion to prevail over the entire cross-section of the pipe, and the damping action named for the solids is absent. For the first stage there is a common exponent which points to a common law governing both, and it is probable that the law is a general one for all stages, the difference in boundary conditions producing the above changes in the exponents.

It will now be shown that there is a common law of resistance applicable to both solids and pipes, or that the general form of the equation applicable to solids, as given in Part I, can also be applied to pipes. There are certain elements which are common to both, such as velocity, diameter, viscosity, and fluid density. They differ in the choice of co-ordinates, V and D being used for the solids and V and S for the pipes, the Δ of the solids corresponding approximately to the S for the pipes.

As this part may be read by some who do not care to read Part I, the origin of the general equation will be fully set forth, and the same method of dimensions will be used.

The Method of Dimensions Applied to Find the Law of Flow for Pipes and Channels.—When generalized, the driving force required to overcome the frictional resistance of a pipe is seen to be the product of a volume, a density, and an acceleration (slope), or, in terms of mass, length, and time,

$$[L^3]\left[\frac{M}{L^3}\right]\left[\frac{L}{T^2}\right] = M\,L\,T^{-2} = \text{A Force} \qquad (14)$$

Opposed to the force and equal thereto is a resistance, which will here be assumed to be a product or function of a velocity, a diameter, a viscosity, and a fluid density, and the additional assumption is made that an exponential relation exists between these four factors, or, that

$$\text{Force} = k\,(V^x\,D^y\,M^z\,\rho^w) \qquad (15)$$

and the coefficient, k, is assumed to be a number not having dimension.

Equation (15), when fully written, becomes

$$[M][L][T^{-2}] = k\Big[[L^x\,T^{-x}]\,[L^y]\,[M^z\,L^{-z}\,T^{-z}]\,[M^w\,L^{-3w}]\Big] \qquad (16)$$

Equating exponents, there is found from

$$
\begin{aligned}
&M, \quad 1 = z + w \\
&L, \quad 1 = x + y - z - 3w \\
&T, \quad -2 = -x - z
\end{aligned}
$$

or, three equations containing four unknown quantities, which can be solved by assuming $n = x = y$ (or that V and D have the common exponent, n). Making this substitution, the resistance becomes

$$\text{Resistance} = k\ (V\ D)^n\ \mu^{2-n}\ \rho^{n-1}$$

from which the general equation can be written

$$k_n\ D^3\ S = (V\ D)^n\ \mu^{2-n}\ \rho^{n-1} \quad \text{.................(17)}$$

This equation is practically identical with that of Part I for solids falling in fluids. Being perfectly general and homogeneous (that is, the sum of the exponents for mass, length, and time are equal in both members of the equation), it offers a sound basis or hypothesis from which to work. It may be objected that the assumptions made about the nature of a resistance are not proved to be true. To this may be said that the assumptions may be fairly considered as true, provided they accord with experimentally determined facts, and such proof will be given later.

The Reynolds Equation.—Professor Osborne Reynolds* proposed an equation for pipes of the form

$$\frac{A\ D^3\ i}{\mu^2} = \left(B\ \frac{D}{\mu}\ V\right)^n$$

in which A and B are coefficients, i is a slope, and μ a viscosity. This equation can also be written

$$A\ D^3\ S = (B\ V\ D)^n\ \mu^{2-n}$$

which differs only from Equation (17) in the absence of the density term, ρ^{n-1}, which is required to make the equation homogeneous and for other reasons shown later. Professor Reynolds' equation never came into use because of the difficulties of assigning proper values to the coefficients and the exponent. It will be shown that A and B are not constants, and, as before stated, n ranges in value from $\frac{5}{3}$ to $\frac{6}{3}$.

* *Proceedings*, Royal Society, 1883.

Special Values of the Exponent, n.—From Equation (17), by giving special values to n, there is found

$$\text{For } n = 1,\ K_1 D^2 S = V \mu \qquad (18)$$

$$\text{For } n = 2,\ K_2 D S = V^2 \rho \qquad (19)$$

$$\text{For } n = 3,\ K_3 S = V^3 \mu^{-1} \rho^2 \qquad (20)$$

Equation (19) is seen to be the basis of the Chezy formula.
Equation (20) can be modified to (making the density = 1)

$$k (S\mu)^{\frac{1}{3}} = V \qquad (21)$$

which form will be used later to find k and to test the general correctness of Equation (17).

Consideration of Equation (17) by Stages.—To test the accuracy of Equation (17) requires that it be applied to experimental data for the three stages, and, if it be correct, then it must agree with experimental results in all three stages.

The First Stage.—

$$K_1 D^2 S = V \mu \qquad (18)$$

For velocities below the lower critical velocity, that at the pipe wall is zero, and calling ϕ a coefficient of roughness, it is clear that flow in the first stage (or, $n = 1$) must be independent of ϕ, hence ϕ can only enter into the general equation with an exponent of $(n - 1)$, or, it is of dimensions the same as a density. Because both ρ^{n-1} and ϕ^{n-1} vanish when $n = 1$, it follows that k_1 must be a constant for any pipe or fluid, and its value is

$$K_1 = \frac{4 V \mu}{D^2 S} \qquad (22)$$

From the ordinary theory of viscosity, it can be shown that in Equation (22) $K_1 = \frac{g}{8}$ or, on reducing,

$$\frac{g D^2 S}{32} = V \mu \qquad (23)$$

Equation (23) is true for any pipe or fluid, and from it is deduced

$$\mu = \frac{g D^2 S}{32 V} \qquad (24)$$

These equations show that all experiments on pipes in the first stage of flow are but experiments for finding the coefficient of viscosity,

and, given the latter, then the velocity can be predicted accurately by the use of Equation (23).

Applying Equation (23) to the reduction of the experiments of Messrs. Saph and Schoder*, which experiments bear internal evidence of having been made with great care, using the centimeter-gramme-second system of units, and Slottes' formula for the coefficient of viscosity of water, there is found for their brass pipes:

Nos.	VII.	IX.	XIII.	XV.	XVI.
log. K_1....	1.4545	1.4830	1.4831	1.4760	1.4852
log. $\frac{g}{32}$.....	1.4866	1.4866	1.4866	1.4866	1.4866
Difference.	—0.0321	—0.0036	—0.0035	—0.0109	—0.0014

These pipes were not jacketed, nor is it known that the viscosity of the water used was that of distilled water; in the case of Pipe No. VII there is only one observation, where the observed loss of head was 0.0315 ft. The small departures noted above can readily fall under experimental errors.

Temperature Effects.—Examining in detail the data from "Brass Pipe No. XVI," there is found for the log. coefficients of K_1 the following values:

Experiment No.	101, $t = 69.0°$	Fahr.,	log. K_1	$= 1.4869$,	Slope	$= 0.1829$
" "	102, $t = 69.5°$	"	" "	1.4942	"	0.0780
" "	113, $t = 70.4°$	"	" "	1.4925	"	0.1172
" "	114, $t = 70.9°$	"	" "	1.4852	"	0.0586
" "	221, $t = 75.2°$	"	" "	1.4888	"	0.0897
Means	$t = 71°$	Fahr.,	log. K_1	$= 1.4888$		

The theoretical value is $K_1 = \log. \frac{g}{32} = 1.4866.$

The air temperature was about 68° Fahr., or nearly the same as that of the water used.

For lower temperatures, there are the following data:

Experiment No.	440, $t = 38.5°$	Fahr.,	log. K_1	$= 1.5026$,	Slope	$= 0.337$
" "	441, $t = 38.4°$	"	" "	$= 1.5178$	"	$= 0.191$
" "	442, $t = 38.3°$	"	" "	$= 1.5487$	"	$= 0.072$

* *Transactions*, Am. Soc. C. E., Vol. LI.

Assuming that log. K_1 is 1.4866, then to harmonize these observations requires changes in the wall temperature, as follows:

t 38.5° should be 40°, difference $= 1.5° \times$ slope $= 0.50$

t 38.4° " " 42°, " $= 3.6° \times$ " $= 0.68$

t 38.3° " " 46°, " $= 7.7° \times$ " $= 0.51$

or the correction required is roughly proportional to the velocity, a high velocity tending to absorb the heat derived from the air temperature difference of about 30° Fahr. These experiments show the necessity for protecting the pipe wall from external temperature changes.

Messrs. Barnes and Coker* have shown that, when a tube is heated from the outside, for velocities below the critical velocity, the heat affects only a mere film of fluid near the wall, as it passes through the tube practically unchanged in temperature. The observations of Messrs. Saph and Schoder are confirmatory of the fact that the surface film is affected. The writer has observed naked pipes in the oil fields of California, conveying oil by gravity, where a passing cloud obscuring the sun would cause a sudden drop in the flow. The action must have been at the pipe surface only, as there was not time to change the temperature of the mass of oil. It can also be inferred that the velocity must have been below the critical velocity. For the higher velocities, $n = \frac{5}{3}$ to $\frac{6}{3}$, it does not appear probable that ordinary differences of temperature could influence the flow greatly, because the water is eddying constantly against the sides and equalizing the temperature. From the foregoing it may be inferred that observations in the $n = 1$ stage are only useful to find the viscosity coefficient, μ, and that the same can be found with much accuracy, provided care is taken to avoid external temperature influences.

Reverting to Equations (22) and (23), it will be observed that their probable origin is from

$$D^3 \frac{\pi}{4} S = \left(\frac{8 \pi V D}{g}\right) \mu \quad \dots\dots\dots\dots (25)$$

The terms, $\frac{\pi}{4}$ and π, are the relations to area and surface, and the 8 is a constant belonging to the terms; it will be shown later that these terms must enter General Equation (17) as previously written.

* *Physical Review*. Vol. XII, 1901.

The foregoing shows that General Equation (17) applies accurately to observations in the ($n = 1$) or first stage of flow.

The Second Stage of Flow.—The end of the first stage of flow (exponent $= 1$) is marked by a rather sudden change in the resistance from the first to the third power. This point is also the lower critical velocity. General Equation (17), making $n = 3$, becomes modified for this stage to its simplest form (making the density $= 1$)

$$k\,(S\,\mu)^{\frac{1}{3}} = V \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

(Note that D disappears from Equation (21).)

Taking Messrs. Saph and Schoder's Brass Pipe No. XVI as a test, the limits of this stage appear to be between

Lower critical velocity, log. $S = 1.4912$, log. $V = 1.8563$
Upper " " log. $S = 1.8563$, log. $V = 1.9780$

and between these limits there are nine observations which, reduced for log. k in Equation (21), give

Experiment No.	99,	log. k	$= 2.6951$
" "	107	" "	$= 2.6971$
" "	109	" "	$= 2.6912$
" "	110	" "	$= 2.6946$
" "	111	" "	$= 2.7054$
" "	112	" "	$= 2.6805$
" "	218	" "	$= 2.6912$
" "	219	" "	$= 2.6871$
" "	317	" "	$= 2.6827$
Mean value			$= 2.6916$

When it is remembered that the experimenters were unaware that such a relation or stage existed, the foregoing very close concordance in the experimental values of the coefficient, k, is to be taken as an index of the care of the experimenters and also as highly confirmatory of the existence of a stage where the resistance varies as the third power of the velocity; at the same time, it confirms the accuracy of General Equation (17) by giving the proper exponent for the viscosity.

The remaining pipes, Nos. VII, IX, XIII, and XV, are not so regular in the values of the coefficient, the reason being that the data

for this region are less in number of observations, and also that some observations are either quite irregular or are too near the critical point. For these pipes the following are approximate mean values:

Pipe	VII,	Mean	log.	$k = 2.744$
"	IX	"	" "	$= 2.762$
"	XIII	"	" "	$= 2.692$
"	XV	"	" "	$= 2.721$
"	XVI	"	" "	$= 2.6916$

The Lower Critical Point.—According to Professor Osborne Reynolds, $\frac{V_c D}{\mu}$ = constant (about 2 000 when using the diameter and centimeter-gramme-second system). It will be shown that this is only partly true. Writing the resistances and equating them for the common point, V_c, there is

$$\frac{V_c D \mu}{k_1} = \frac{(V_c D)^3}{k_3 \mu}$$

from which is found

$$\left(\frac{k_3}{k_1}\right)^{\frac{1}{2}} = \frac{V_c D}{\mu}$$

an equation which shows that $\frac{V_c D}{\mu}$ can only be a constant provided that k_3 is constant (k_1 is a constant), but the above values of k_3, which is the above k cubed, are not constant, which suggests that k_3 must contain $(\phi\ \rho)^2$, or $(\phi\ \rho)^{n-1}$, because it can be assumed that it is the roughness factor, ϕ, that determines the change in the law of resistance from V_c to V_c^3. For water, $\rho = 1$, hence ϕ^2 must be a factor of k_3, or $\phi^{\frac{2}{3}}$ is a factor of the k of Equation (21).

Consideration of k_n of Equation (17).—At this stage it is appropriate to consider the nature of the general coefficient, k_n, of Equation (17). From a study of the coefficients found in the three stages of flow, it became clear that k_n contains as factors, g^n, $\frac{1}{n\,(n-1)}$, $(8\,\pi)^n$, and $(\rho\ \phi)^{n-1}$. Further consideration of these factors, in connection with Equations (17) and (25), led to the following general form, which is Equation (17) more fully detailed:

$$D^3 \frac{\pi}{4} S = \frac{1}{n\,(n-1)} \left(\frac{8\,\pi\,V D}{g}\right)^n \cdot \mu^{2-n} (\rho\ \phi)^{n-1} \ldots\ldots (26)$$

in which form it may be considered as applicable to any fluid. If the constants, $\frac{8\,\pi}{g}$, are removed from the bracket, or are transposed to the left-hand member, then the term, ϕ^{n-1}, is no longer single-valued. The term, $\frac{1}{n}$, is supposed to be an integration coefficient; the term, $\frac{1}{(n-1)}$, is also required to render ϕ^{n-1} a single-valued function, when $n = 3$ or n. Equation (26) can also be written

$$D^3 \frac{\pi}{4} S \frac{\rho\,\phi}{\mu^2} = \frac{1}{n\,(n-1)} \left(\frac{8\,\pi\,V\,D\,\phi}{g\,\mu}\right)^n.$$

In this form it is seen that the terms, μ^{2-n}, $(\rho\,\phi)^{n-1}$, do not have or require integration coefficients, other than the term, $\frac{1}{n\,(n-1)}$, common to both equations. From this it is seen that, for a given pipe in the third, or n, stage of flow, V varies as

$$\mu^{\left(\frac{2-n}{n}\right)} \quad \text{.........................} \quad (27)$$

and temperature reductions are made accurately by using this exponent for μ. As an illustration, let $n = 1.75$, log. $V = 0.6000$, and $t = 35°$ Fahr.; required, log. V at $t = 70°$ Fahr.

For $t = 35°$ Fahr., log. $\mu = \overline{2}.2286$

For $t = 70°$ Fahr., log. $\mu = \overline{3}.9878$

difference $= 0.2408$

$\frac{2 - 1.75}{1.75} = \frac{1}{7}$ difference $\quad 0.0344$

0.6000 log. V at 35° Fahr.

0.6344 log. V at 70° Fahr.

or 8.25% increase in the velocity from such temperature change.

Further Consideration of the Critical Points.—Making $n = 1$ and $n = 3$ for the first and second stages, using the final Equation (26), and equating resistances for the two stages and common lower V_c point, there is

$$\left(\frac{8\,\pi\,V_c\,D}{g}\right)\mu = \frac{1}{6}\left(\frac{8\,\pi\,V_c\,D}{g}\right)^3 \mu^{-1}\,\phi^2$$

from which is found

$$95.63\,\mu = V_c\,D\,\phi \quad \text{.........................} \quad (28)$$

from which it is seen that $\frac{V_c\,D\,\phi}{\mu}$ = constant; and the statement of Professor Reynolds is modified by the introduction of the factor, ϕ.

The Upper Critical Point.—Making $n = 3$ and n, for the second and third stages, equating resistances for the common upper V_c point, by Equation (26), and reducing, there is found

$$\left(\frac{6}{n\,(n-1)}\right)^{\frac{1}{3-n}} \frac{g\,\mu}{8\,\pi} = V_c\,D\,\phi \ldots\ldots\ldots\ldots (29)$$

which shows the relations existing between the factors at the upper V_c, or beginning of the third stage.

As ϕ does not vary greatly for the brass pipes, the effect of both Equations (28) and (29), when using a logarithmic diagram and a common temperature, is to make both the lower and upper V_c points lie very nearly on a straight line, as noted by Messrs. Saph and Schoder.

From this it is seen that the factor, ϕ, is the determining cause of the change in the exponents; also, it can be inferred that the smoother and more regular the pipe wall the more sudden will be the change of exponents. Just why there is a change from the first to the third power in the resistance is not known, but the following is offered as a tentative solution.

Up to the lower V_c the fluid is assumed to be moving in parallel filaments, and there is no motion at the pipe wall. When the V_c point is reached, the motion is unstable and suddenly becomes turbulent, when the kinetic energy of the fluid is released and begins to expend itself in doing internal work which disappears as heat. As the kinetic energy is as V^2, this, by V, becomes V^3. Possibly in this case the kinetic energy might be represented (on a logarithmic diagram) by a triangle with a vertex at the upper V_c, all the surplus kinetic energy due from the lower V_c having been used up, and the new regime of $V^{n\text{th}}$ begins. This reasoning finds support in the fact that D^3 cancels out in both sides of Equation (26), thus showing that it is not dependent on the diameter, but on the kinetic energy present at the lower V_c. This stage, aside from its value as an easy method of finding the value of ϕ, when the pipe is not too large, would also have great value in finding the differential equation, on which Equation (26) must be founded.

Experimental Values of n, V_c, V_0, and ϕ.—Applying Equation (26), and using as exponents, 1, 3, and n, to the reduction of the experimental data of Messrs. Saph and Schoder, the following values are found:

Small changes in the value of the exponent are found, depending on the grouping of the observations. As these experiments extended over some time, it is seen that there are probably changes in the surface conditions of the pipes sufficient to produce a few hundredths in the value of the exponent. For the reduction given here, such values for finding n were used as were nearest in their numbers to those given for the $n = 3$ stage. Thus, for Pipe VII, Observations 56, 199, 201, and 388 were used for $n = 3$, and Observations 381, 382, and 383, with Observations 390, 391, 392, and 393, to find n; and for Pipe XVI, Observations 99, 107, 109, 110, 111, and 112, were used for $n = 3$, and Observations 103 and 104, with Observations 105 and 106, to find n.

TABLE 3.—VALUES OF n, ϕ, AND THE V_c POINTS.

Pipe No.	log. D	Exponent, n.	Lower Critical Point. $t = 70°$ Fahr.			Upper Critical Point. $t = 70°$ Fahr.			Intersection of $n = 1$ with $n = n$, or $\times$ of Fig. 1. $t = 70°$ Fahr.		
			log. S	log. $V_c D$	log. ϕ	log. S	log. $V_c D$	log. ϕ	log S	log. $V_c D$	log. ϕ
VII...	0.2042	1.7783	$\overline{3}.2492$	1.3607	$\overline{2}.6417$	$\overline{3}.7089$	1.5139	$\overline{2}.6441$	$\overline{3}.0087$	1.1202	$\overline{2}.6441$
XVI..	$\overline{1}.4338$	1.8263	$\overline{1}.4849$	1.2852	$\overline{2}.6831$	1.8512	1.4082	$\overline{2}.6817$	$\overline{1}.3135$	1.1138	2.6817

Table 3 shows a close agreement in the two values of ϕ found for the second and third stages, and it confirms the general arrangement of Equation (26), which is the only method found that makes ϕ of single value in the two stages.

To give a clear idea to those to whom the researches of Messrs. Saph and Schoder are not readily available, reference is made to Fig. 1, which shows the position on a logarithmic diagram of the three stages, $n = 1$, 3, and 1.794, for temperatures of 70° and 40° Fahr. These lines were obtained by Equation (17) and substituting therein the foregoing values of n, which gave the coefficients, k, k_3, and k_n; with these values, the intersection points, A, B, and C, D, were computed.

It will be noted that the point, C, is of much greater velocity than the point, A. Equation (28) shows that V_c varies directly as the viscosity. The equation shows that the $t = 40°$ or $C — D$ line is gen-

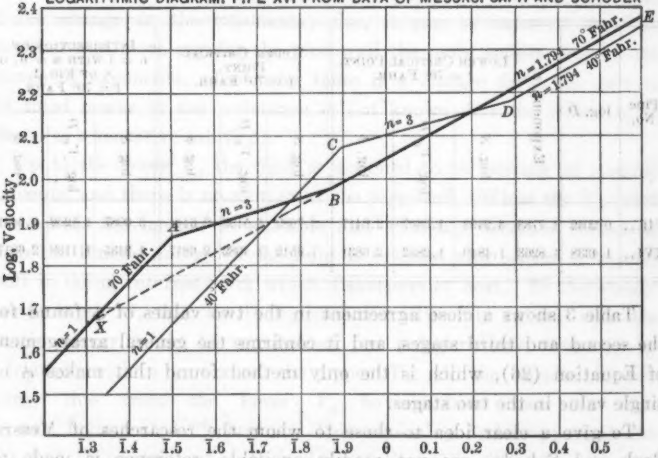
erated by the motion of the $t = 70°$ or $A - B$ line along the line, $A - C$, which has a slope of log. $S = 2$, log. $V = 1$.

Log. Slope.

Point	$A =$ log. $S = \bar{1}.4912$	log. $V = 1.8563$
"	$B =$ log. $S = \bar{1}.8563$	log. $V = 1.9780$
"	$C =$ log. $S = \bar{1}.8928$	log. $V = 2.0571$
"	$D =$ log. $S = 0.2579$	log. $V = 2.1788$

70° log. viscosity	$\bar{3}.9877$
40° " "	$\bar{2}.1885$
Difference	$= 0.2008$

LOGARITHMIC DIAGRAM. PIPE XVI FROM DATA OF MESSRS. SAPH AND SCHODER



Log. Slope.

FIG. 1.

The Third Stage of Flow, V^n.—This stage is the most interesting from a practical viewpoint, as it is the one met in daily use. General Equation (26) represents quite accurately the experimental data, and it may be well to state here the method used to find the exponent, n, and the proof of the accuracy of the term, μ^{2-n}.

Calling *a.c.* the arithmetical complement, write a new equation, as follows:

$$x\,(a.c.\ \log. S) + \log. V - y\,(a.c.\ \log. \mu) = \text{Constant} \ldots (30)$$

On the application of this equation to all the data of a given pipe above the upper critical velocity, the solution will be $x = \frac{1}{n}$, and y will equal a value very close to $\frac{2 - n}{n}$; thus, for $n = 1.75$ to 1.80, small differences of 0.01 to 0.02 were found from the value of $2 - n$. Averaging all the data of Messrs. Saph and Schoder for their brass pipes, II to XVI, inclusive, it was found that the statement V varies as $\mu^{\left(\frac{2-n}{n}\right)}$, is correct. Some larger differences than those above noted were found, and it seemed to point to progressive changes in the condition of the pipe surface. Those observations which were nearest to each other in time gave more concordant results than those found by comparing old and new observations. Differences in the exponent and also for ϕ were noted. The general impression left on the writer's mind was that, for purely experimental purposes, such as to discover the laws of flow, greater care must be taken with the experiments, possibly in the matter of jacketing the pipe, so as to have the wall temperature the same as the water moving in the pipe, and also that the water be quite free from sediment and bacterial or organic growths which might lodge on the rugosities of the surface, or could fill any pits in the surface, or, in the case of organic growths, might in a short time change seriously the surface condition to an extent detrimental to the accuracy of the experimental data sought. Filtration of the water and its sterilization would seem to be indicated as necessary requirements for any approach to accuracy.

Results Obtained by Equation (26).—Table 4 gives values of D, n, and ϕ, as found by applying Equation (26) to the reduction of some of the existing pipe data. With the exception of the experiments of Messrs. Saph and Schoder, most of the data lack details as to temperature. In order to make use of some of it, the temperature was assumed, as it seemed especially desirable to compare the exponents found for wood pipe with the corresponding values of ϕ for brass pipe.

Table 5 gives the viscosity coefficient, as used by the writer, to enable comparisons to be made by those to whom such tables are not readily accessible.

TABLE 4.—Values of Diameter, Exponent, and the Coefficient, ϕ.

Pipe No.	log. D, in centimeters.	Exponent n.	ϕ	Remarks.
Saph and Schoder—Brass Pipes.				
II.	0.7202	1.7600	0.0445	
III.	0.5803	1.7611	0.0476	
IV.	0.5005	1.7540	0.0467	
V.	0.4277	1.7547	0.0465	
VI.	0.3185	1.7380	0.0456	
VII.	0.2042	1.7618	0.0453	
IX.	$\bar{1}.9805$	1.7749	0.0445	
XIII.	$\bar{1}.7502$	1.7417	0.0479	
XV.	$\bar{1}.5885$	1.7440	0.0444	
XVI.	$\bar{1}.4338$	1.7940	0.0466	

Saph and Schoder—Galvanized Pipes.

Pipe.		Exponent.	log. ϕ	
XVII direct	0.4214	1.9815	$\bar{2}.7110$	
XVII reverse	0.4214	1.9353	$\bar{2}.7579$	
XVIII direct	0.3342	1.8292	$\bar{2}.6840$	
XVIII reverse	0.3342	1.8533	$\bar{2}.6673$	
XIX direct	0.2014	1.8420	$\bar{2}.7513$	
XIX reverse	0.2014	1.8676	$\bar{2}.7281$	
XX direct	0.0914	1.9186	$\bar{2}.7348$	
XX reverse	0.0914	1.9084	$\bar{2}.7457$	
XXI direct	$\bar{1}.9489$	1.9593	$\bar{2}.9061$	
XXI reverse	$\bar{1}.9489$	1.9302	$\bar{2}.9471$	

Diameter, in inches.	log. D, in centimeters.	Exponent, n.	ϕ	Remarks.
E. A. Moritz—Wood Stave Pipes, t Assumed at 55° Fahr.				
55.75	2.1514	1.746	0.0610	
55.75	2.1514	1.757	0.0630	Year, 1909
22.00	1.7472	2.215	0.013	" 1910
18	1.6601	1.758	0.0582	
14	1.5509	1.863	0.0434	
4	1.5509	1.868	0.0399	" 1910
2	1.4840	1.749	0.1036	" 1909
8	1.3079	1.748	0.0538	
6	1.1830	1.794	0.0528	
6	1.1830	1.850	0.0508	" 1909
5	1.1038	1.896	0.0593	" 1910
4	1.0069	1.698	0.1144	

TABLE 4—(*Continued*).

Diameter, in inches.	log. D. in centimeters.	Exponent, n.	ϕ	Remarks.
J. S. Moore—Wood Stave Pipe.				
48.75	2.0928	1.778	0.0568	
31	1.8962	1.770	0.0598	
Desmond FitzGerald—Cast-Iron Pipe.				
48	2.0858	1.9696	0.0340	North pipe tuberculated.
48	2.0861	1.8787	0.0310	Cleaned.
Marx, Wing, and Hoskins.				
72.25	......	1.945	0.0410	Steel.
72.25	......	1.939	0.0300	Wood.
Clemens Herschel.*				
48	......	1.856	0.0702	
Clemens Herschel—Holyoke Conduit.				
103.38	......	2.08	0.0129	Iron.

* "115 Experiments," p. 27, Experiment 36, "Cylindrical Joints."

TABLE 5.—Common Log. of the Viscosity Coefficient,

$$\mu = \frac{0.017944}{(1 + 0.023121\ T)^{1.5423}}$$

(T = Centigrade degrees.)

t. Fahr.	log. μ	t. Fahr.	log. μ	t. Fahr.	log. μ	t. Fahr.	log. μ
32°	$\bar{2}.2539$	48°	$\bar{2}.1287$	64°	$\bar{2}.0233$	80°	$\bar{3}.9321$
33°	$\bar{2}.2454$	49°	$\bar{2}.1216$	65°	$\bar{2}.0172$	81°	$\bar{3}.9267$
34°	$\bar{2}.2369$	50°	$\bar{2}.1146$	66°	$\bar{2}.0112$	82°	$\bar{3}.9213$
35°	$\bar{2}.2286$	51°	$\bar{2}.1076$	67°	$\bar{2}.0052$	83°	$\bar{3}.9160$
36°	$\bar{2}.2204$	52°	$\bar{2}.1007$	68°	$\bar{3}.9993$	84°	$\bar{3}.9110$
37°	$\bar{2}.2122$	53°	$\bar{2}.0937$	69°	$\bar{3}.9935$	85°	$\bar{3}.9060$
38°	$\bar{2}.2042$	54°	$\bar{2}.0871$	70°	$\bar{3}.9877$	86°	$\bar{3}.9011$
39°	$\bar{2}.1963$	55°	$\bar{2}.0805$	71°	$\bar{3}.9819$	87°	$\bar{3}.8962$
40°	$\bar{2}.1885$	56°	$\bar{2}.0739$	72°	$\bar{3}.9762$	88°	$\bar{3}.8914$
41°	$\bar{2}.1807$	57°	$\bar{2}.0674$	73°	$\bar{3}.9705$	89°	$\bar{3}.8866$
42°	$\bar{2}.1731$	58°	$\bar{2}.0609$	74°	$\bar{3}.9649$	90°	$\bar{3}.8806$
43°	$\bar{2}.1655$	59°	$\bar{2}.0545$	75°	$\bar{3}.9593$	91°	$\bar{3}.8758$
44°	$\bar{2}.1579$	60°	$\bar{2}.0481$	76°	$\bar{3}.9538$	92°	$\bar{3}.8709$
45°	$\bar{2}.1505$	61°	$\bar{2}.0418$	77°	$\bar{3}.9483$	93°	$\bar{3}.8662$
46°	$\bar{2}.1432$	62°	$\bar{2}.0356$	78°	$\bar{3}.9428$	94°	$\bar{3}.8615$
47°	$\bar{2}.1359$	63°	$\bar{2}.0295$	79°	$\bar{3}.9374$	95°	$\bar{3}.8568$

Comparison of Equation (26) with the Usual Forms.—Calling C a variable coefficient which contains ϕ and μ, Equation (26) can be reduced to

$$C\, D^{\left(\frac{3}{n}-1\right)}\, S^{\frac{1}{n}} = V \quad \ldots\ldots\ldots\ldots\ldots\ldots (31)$$

Making $n = 2$, Equation (31) is the Chezy form; otherwise it is the ordinary exponential equation as used by various writers. It was suggested by the late Charles H. Tutton, M. Am. Soc. C. E., and others that the sum of the exponents in the terms, D and S, are constant and equal to 1.17. This is only true when $n = \dfrac{3+1}{1.17+1} = 1.843$, or, their $R^{0.63}\; S^{0.54}$. Messrs. Saph and Schoder give for brass pipes $n = 1.75$, which is $D^{\frac{5}{7}}\, S^{\frac{4}{7}}$, or the same as their $D^{0.71}\; S^{0.57}$. Mr. J. S. Moore,* for wood stave pipes, gives $H = \dfrac{8.3\, V^{1.78}}{D^{1.20}}$, which is $1.78 + 1.20 = 2.98$, instead of 3, as required in Equation (31). As the sum of the exponents must be equal for each member of an equation, it is seen that when $n = 1.75$, then the C of Equation (31) requires an exponent of $\left(2 - \dfrac{4}{n}\right)$ or, $-\,0.285$, to satisfy the condition of equality of the sum of the exponents; which shows the futility of any attempts at precision unless the actual n, ϕ and μ be used, as set forth in Equation (26).

Recapitulation by Stages.—General Equation (26) is shown to apply quite accurately to the first stage ($n = 1$), and its derivative, Equation (23),

$$\frac{g\, D^2\, S}{32} = V\, \mu$$

is applicable to any pipe or fluid.

For the second, or V^3, stage, the general equation accords very closely with experiment. The change of the exponent of μ from $+\,1$ in the first stage, to $-\,1$ in the second stage, is brought out. By its use the apparently erratic observations of Messrs. Saph and Schoder over the critical region of flow are shown to accord quite closely with theory.

For the third, or the V^n, stage, the general equation makes use of the actual exponent, and also makes V vary as $\mu^{\frac{(2-n)}{n}}$, which form of exponent for μ agrees very closely with the experiments.

* *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 471.

Lastly, ϕ is a single-valued factor, with exponents of 2, and $(n - 1)$ in the second and third stages.

From which it can be said that the general equation found by dimensional methods is capable of satisfying all the known experimental data, and may be considered as correct.

The Exponent, n, and Roughness Factor, ϕ.—These are interdependent quantities, and, before any general use can be made of Equation (26), some method for their evaluation must be found. The general relation existing between these factors will next be discussed, and this will be followed by suggestions for further research work required for their definite evaluation.

The Relation of the Variables, n, ϕ, and V_0.—It has been shown that for $n = 1$, the coefficient, k, is $\frac{g}{32}$, a constant, and it is also clear that the diameter, D, and the viscosity factor, μ, can also be made constant, thus giving for the first stage a constant reference line, to which line the third stage can be prolonged, the intersection point being the foregoing V_0, marked X on Fig. 1. Although this point has no actual physical existence, it is a useful assumption, as it throws light on the genesis of $(n - 1)$. Making $n = 1$ and n, and equating resistances for each, as given by Equation (26), there is

$$\left(\frac{8\,\pi\,D\,V_0}{g}\right)\mu = \frac{1}{n\,(n-1)}\left(\frac{8\,\pi\,V_0\,D}{g}\right)^n \mu^{2-n}\,\phi^{n-1} \ldots (32)$$

which reduces to

$$n\,(n-1)^{\frac{1}{n-1}} = \frac{8\,\pi\,V_0\,D\,\phi}{g\,\mu} \ldots\ldots\ldots\ldots (33)$$

Make $D = 1$, let μ be a constant (say for 70° Fahr.), and make $\frac{8\,\pi\,D}{g\,\mu} = 2\,b$, and Equation (33) becomes

$$n\,(n-1)^{\frac{1}{n-1}} = 2\,b\,V_0\,\phi \ldots\ldots\ldots\ldots (34)$$

Writing over each member of Equation (34) the equality $(n - 1)$, there is found

$$\frac{n-1}{n\,(n-1)^{\frac{1}{n-1}}} = \frac{n-1}{2\,b\,V_0\,\phi} \ldots\ldots\ldots\ldots (35)$$

The left-hand member of Equation (35) has the remarkable property that it is practically equal to $\frac{1}{2}$, as here shown.

Values of the Exponent and Function, n, of Equation (35).—

	$n = 2$	1.95	1.90	1.85	1.80	1.75
function	$n = 0.5000$	0.4965	0.4959	0.4990	0.5071	0.5231

Hence, for the usual range of exponents, $\frac{1}{2}$ may be written for function n, from which is obtained

$$(n - 1) = b\ V_0\ \phi \dots\dots\dots\dots\dots\dots (36)$$

an equation, very approximately correct, which can be made exact by using the proper value of function n instead of $\frac{1}{2}$.

Equation (36) shows that the term $(n - 1)$, or the fractional part of the exponent, n, is made up of a constant, into the limiting velocity, V_0, and the factor, ϕ, supposed to be either the absolute roughness factor, or a function of such factor.

Reverting to Equation (34), substitute for $(n - 1)$ the value, $b\ V_0\ \phi$, and there is found

$$(b\ V_0\ \phi)^2 + b\ V_0\ \phi = (2\ b\ V_0\ \phi)^{b\ V_0\ \phi} \dots\dots\dots\dots (37)$$

an equation which may serve to throw some light on the interdependent relations of V_0 and ϕ. If the assumption be made that the exponent, n, is never greater than 2, or that 2 is the upper limit for pipes and channels, which assumption appears to be true as judged by the facts, then it follows that a probable relation is

$$(n - 1) = b\ V_0\ \phi = \tanh.\ u \dots\dots\dots\dots (38)$$

where u might be considered some measure of the surface condition into $b\ V_0$, or

$$u = \frac{1}{2} \log._e \left(\frac{1 + b\ V_0 \phi}{1 - b\ V_0\ \phi}\right) \dots\dots\dots\dots (39)$$

Concerning Equations (33) to (39), it is the writer's impression that any attempt at present to separate the variables would be useless, for the reason that there is now no method known of stating the numerical value of a roughness of a surface, and if the absolute value of ϕ, or such a roughness, could be deduced from existing data, it would still require further experimental work to connect such a value with a given surface.

For comparative purposes, four exemplars are here given. The mean value of V^n for some corresponding S, was reduced to $t = 70°$

Fahr., and using Equation (26) with $n = 1$ and n, the values of V_0 and ϕ were computed.

No. 1 is the average values for Messrs. Saph and Schoder's brass pipes, II, III, IV, V, VI, and VII, and Nos. 17, 18, and 21, are for their galvanized pipes.

No.	1	17	18	21
n	1.7547	1.9404	1.8286	1.9525
log. V_0	1.0766	1.1073	1.1115	0.9510
log. ϕ	$\bar{2}.6643$	$\bar{2}.7497$	$\bar{2}.6858$	$\bar{2}.9111$
log. $V_0\ \phi$	$\bar{1}.7409$	$\bar{1}.8570$	$\bar{1}.7973$	$\bar{1}.8621$
log. u	$\bar{1}.9929$	0.2409	0.0732	0.2691
log. $\frac{u}{V_0}$	$\bar{2}.9163$	$\bar{1}.1336$	$\bar{2}.9617$	$\bar{1}.3319$
Relative roughness	1.	1.65	1.11	2.61

The quantity, u, is the corresponding argument in a table of hyperbolic tangents that gives, tanh. $u = (n - 1)$. It will be noted that for Nos. 17 and 21, $V_0\ \phi$ is nearly a constant, hence V_0 is some reciprocal function of ϕ. An inspection of its relation to the other quantities shows that it is probably transcendental and closely related to the relative roughness.

The last line, relative roughness, may be taken as an expression for the roughness, calling brass pipes $= 1$. It is this measure that is required in absolute measure, and its relation to V_0 and ϕ is required to effect the solution of Equation (36).

Probable Factors of the Exponent, n.—Equations (32) and (39) suggest the thought that the changes in the exponent for the three stages of flow have their origin in changes of boundary conditions, or certain limiting velocities combined with the surface condition, producing the observed changes, and, before proceeding to build up any theory of the cause of such changes, a brief review will be made of the possible contributing factors.

The Experimental Value of n.—For pipes and channels, our knowledge of the value of the exponent is derived from the relation

$$n = \frac{\log. S_1 - \log. S}{\log. V_1 - \log. V} = \frac{d \log. S}{d \log. V} = \frac{V\, d\, s}{S\, d\, V} \quad \text{........(40)}$$

General Equation (16) assumes that the resistance is a function of the surface, or that the surface condition determines the resistance

and its exponent, and Equation (40) shows that the exponent is found from a loss of head, and it should here be noted that there can be, and in practice usually is, loss of head not directly chargeable to the general surface condition, such as variations in diameter, as a cylindrical-jointed or a taper-jointed pipe; from the grosser inequalities of the surface, such as a row of rivet heads, circular or longitudinal, corrugations in the asphalt dip, large tubercles; any change from straight-line motion, such as bends in the pipe; leakage, air at the summits, etc. For all these abnormal conditions it is possible that the resistances follow some other law of exponent than that due to the general surface conditions, such as the resistance for the abnormal conditions might follow the V^2 law, or some law approximating it, and the normal resistance, such as is supposed to arise from a general surface condition, might be taken as the V^n law. This is a very important point, not heretofore recognized, and before any accurate advance in the theory can be made, means must be discovered for the separate evaluation of normal and abnormal losses of head.

As this paper is dealing with normal conditions, it will consider that the experimental results of the best selected data show that n ranges from $\frac{5}{3}$ to $\frac{6}{3}$ in magnitude.

The Exponent Increases with the Roughness.—Smooth pipes have exponents of 1.70 to 1.80, and rough pipes have exponents of 1.90 to 1.99 +, but do not reach the upper value of 2.

Velocity as a Factor.—The investigations of both Parts I and II, show that sudden changes occur in the value of the exponent, and that such changes are caused by, or at, a certain critical velocity, and that, on a logarithmic diagram, the critical points are connected by straight lines from point to point. As far as can be judged from the examination of the data, the above-mentioned straight lines are not asymptotes, but the change appears to arise from a condition of unstable equilibrium, much like a balance just poised, when a very small addition to the weight will cause the beam to drop. It should here be remarked that the evidence on this point is by no means conclusive; thus, if careful experiments should show that the straight lines are really connected by a short curvature, it would necessarily modify the above-named opinion that the critical points are connected by straight lines, and might also modify the mathematical treatment of the problem.

Is the Diameter a Factor of the Exponent?—Equation (36) shows that the diameter is not a factor, because the b can be considered a constant; if some other diameter than unity be used, then the value of b would be correspondingly changed, with a simultaneous and equal change in V_0, or, $b_1\ V_{01}\ \phi = b\ V_0\ \phi = (n - 1)$. Table 4 shows that for brass pipes ranging from 0.1 to 2 in. in diameter, n and ϕ remain nearly the same through a range of 1 to 20; again, compare these brass pipes with the large wood stave pipes of Messrs. Moritz and Moore, where the exponents and ϕ are almost the same as for the brass pipes, or, there is practically the same n and ϕ through a range of 575 diameters, from which it may fairly be concluded that the exponent is independent of the diameter. A logical conclusion from which is, that, for the purpose of finding laws experimentally, a small pipe is more effective than a large one, because better control of all the details can be had.

The Exponent, n, is Independent of Viscosity.—The b of Equation (36) contains the viscosity factor, and, reducing all the experiments to a constant temperature, there still remain variations of V_0 and ϕ. Experimentally, this statement is confirmed as follows:

Fifteen brass pipes experimented with by Messrs. Saph and Schoder (see their Table No. 6) gave as a mean average value

$$t, 70^\circ \text{ Fahr. log. } \mu = 3.9877 \quad n = 1.7445$$
$$t, 40^\circ \text{ Fahr. log. } \mu = 2.1885 \quad n = 1.7438$$
$$\text{difference, } 30^\circ \text{ Fahr. log. } \mu = 0.2008 \quad n = 0.0007$$

or, a difference of 1.587 times in the value of the viscosity coefficient does not practically change the value of the exponent. Again, if, on a logarithmic diagram, the points having common temperatures and different velocities be connected, a series of parallel lines results. All of which shows that the viscosity factor is only a coefficient the exponent of which changes as set forth in Equation (17). The above paragraph is quite important in respect to the conclusions of the following paragraph.

Change of Exponent from a Change of Fluid.—Experiments made by A. M. Hunt,* M. Am. Soc. C. E., with 4- and 6-in. screwed steel

* *Journal of Electricity, Power and Gas*, January, 1905.

pipes, used for conveying crude oil from the Coalinga, Cal., oil field, gave for the values of n

$$\text{4-in. pipe, } n = 1.485$$

$$\text{6-in. pipe, } n = 1.132$$

Nothing was said about temperature or viscosity. A bulletin issued by the California State Mining Bureau gives for average Coalinga crude oil

Viscosity, t 15° cent., 3.09 times greater than for water.
" t 85° cent., 1.17 " " " " "

The only point desired to be brought out is that there is a very great change in the exponent due to a change of fluid. For water, these pipes would have had exponents of about 1.85 to 1.90. Assuming that the viscosity is three or four times greater than that for water, compare it with the more accurate data of Messrs. Saph and Schoder, where a difference of log. $\mu = 0.2008$ gave a difference of only 0.0007 in the value of n. Accepting Mr. Hunt's data as substantially correct, some other explanation than that of greater viscosity must be had for such large changes in the exponent. One possible explanation is that it is due to some other property of the fluid molecule than viscosity, such as atomic weight, molecular diameter, mean free path, form of molecule, etc. As the atomic weight for oil is greater than for water, it appears that this property must operate to reduce the exponent. Provisionally, it will be called $(a\ t)$ and as such will appear among the probable factors of the exponent, n, as, $(a\ t)^{-z}$, the exponent $-z$, being introduced, as it is at present unknown. Direct experiment with different fluids would probably give a constant for each fluid of the form, const. $^{-1}$.

Possible Change of Exponent by Reversal of Flow.—This matter is somewhat uncertain, because, as previously stated, of the time that elapsed between experiments where there might have been progressive changes of the surface conditions, but, on the whole, the evidence points to such a change. It is conceivable that, in the process of manufacture of a pipe, the surface inequalities might have a greater average slope in one direction than another, or, this combined with the pattern arrangement might, on a reversal of the direction of flow, cause small changes in V_0 and ϕ, hence there would be changes in the exponent. At present, the information on this point does not warrant any fur-

ther notice than a general caution to future experimenters to consider the matter.

The Roughness Factor.—The inner surfaces of seamless brass tubing (which to the eye were quite smooth), when examined under a microscope having a power of 60 diameters, showed a surface which appeared to be covered with small wart-like masses separated by dark lines. No idea of their height was gained. The diameters varied from 0.002 to 0.004 cm., with an average of about 0.0035 cm. Galvanized pipes were more irregular. A pipe 1 in. in diameter had one side of about the same degree of roughness as that of the iron from which the zinc had been stripped. The other side was far rougher, and was quite drossy; elongated tears of zinc showed that this was probably the bottom side while draining. The average diameter of the roughnesses was from 0.01 to 0.02 cm., averaged at 0.016 cm. A $\frac{1}{2}$-in. galvanized pipe was quite smooth, and nearly comparable to the brass pipes. A $\frac{3}{8}$-in. galvanized pipe was found to be rough and angular, with a quite different grouping from the others. The average diameter was about 0.01 to 0.02 cm., and it was apparent that the rugosities were of greater height. It was the writer's impression, from this cursory examination, that by using photo-micrographs (or other suitable methods), the differences noted could be expressed in definite terms, and perhaps correlated with the exponent. It was made quite clear that there are minute differences which affect the exponent and coefficient, ϕ, and are not visible to the unaided eye.

On a Definition of a Roughness.—At present, the only means of expressing the degree of roughness is adjectival. In default of exact knowledge, the terms, rough, rougher, etc., are used, when what is really required is a definite numerical value, as an essential to any further advance. The foregoing examination seems to indicate that there is a possibility of obtaining a numerical definition by noting the average number of rugosities per unit surface, their mean height, and general cross-section. In addition to these general features, it is probable that the pattern arrangement might be such as to influence the stream-line motion of the fluid.

Broadly stated, a numerical definition would consist of the average departure from a perfect surface, combined with complete knowledge of the form and arrangement of an average departure, which in turn suggests the following as measurable quantities:

N_2, the number of rugosities per unit surface.

h, the average height of the rugosity.

It is possible that this might be sufficient for practical purposes, but for scientific purposes, a further classification might be required, such as,

θ, an average angle of one rugosity with the pipe axis.

λ, an average angle of rugosity to rugosity referred to the pipe axis (pattern arrangement).

The two latter factors would certainly be required if there should be found that the succeeding paragraph is a necessary factor of flow.

General Remarks on Turbulent Flow.—Exploration by Pitot tubes of the velocities over the area of a pipe in turbulent flow shows that the velocity near the wall is about one-half of that at the center of the pipe, and the mean velocity is about 0.68 of the radius from the center, the central and wall velocities being joined by a parabolic or elliptical curve. This statement, taken singly, implies that the central part of the fluid tends to run away from the fluid nearer the wall, or a state of non-diffusion must exist. Opposed to this view are the actual experiments of Benzenberg, Campbell, and others, where dye, bran, etc., have been used to measure the mean velocity, the underlying principle being, that there is such diffusion, accompanied by translation, and that after a long run, the marking material may still be considered as a fairly compact body, which serves as an accurate measure of the mean velocity. Such experiments taken singly show an active circulation across the pipe, in some manner, by which the marking substance diffuses transversely, but not longitudinally.

The experiments of Reynolds show that the beginning of turbulent motion is a spiral flow of the marking substance. From all three statements it is probable that the fluid motion is checked at the wall, diffuses toward the center, and then returns toward the wall; at the same time it is subject to longitudinal translation. Hence it is not improbable that the mean motion is at the same time spiral; or, all fluid molecules tend to rotation about the pipe axis. Hence it follows that Pitot tube measurements must be those of one component of the mean motion, the actual velocity of the moving fluid being greater and generally at an angle with the pipe axis. It thus appears that

the actual motion or average velocity of a point may be something quite different from the mean flow or resultant of all the points.

There is a marked difference between the stream-line motion of a solid moving in a fluid and that of a fluid moving in a pipe, and it should be expected that the nature of the stream function should be different. For the case of turbulent motion (or exponents of resistance greater than one), the maximum disturbance of the fluid molecule must be near the moving solid and due to viscosity; this motion is damped and completely dies away at some distance normal to the axis of motion. In the case of turbulent flow in a pipe, there is no such damping action. All the molecules are disturbed in varying degree over the entire area of the pipe section.

If the foregoing assumption of a mean spiral path of a particle be correct, then the mean motion is periodic, and a given particle would tend to reappear more or less in the same place, measured around the pipe, the recurring points separated along the pipe axis by a period of one complete rotation. One particle, however, must react on the next nearest to it, and, in the supposed case of general spiral motion, it is not improbable that there might be superposed on the general spiral path an oscillatory motion, also periodic in its nature, the latter motion probably corresponding to a dissipation of energy function and the former to a stream-line function.

At present, nothing is known of the actual motion in a pipe, and the foregoing paragraph is only suggestive of a factor that may require estimation for the complete development of Equations (32) to (36). For the actual inquiry, it seems that by the use of a glass pipe carrying fluid charged with opaque particles of about the same density, and the use of a camera traveling at the mean velocity, it would be possible to record accurately and trace out the path of one or more opaque particles.

Recapitulation.—The initial stage of flow has 1 for exponent, and it suddenly becomes n, hence the problem is to find the cause of the addition of $(n - 1)$. Collecting from the preceding discussion the possible factors of $(n - 1)$, there is found in the order given, the following list of probable factors of $(n - 1)$:

a, magnitude, ranges from $\frac{3}{4}$ to 1;
b, roughness, ϕ increases with the roughness;
c, critical velocity, V_0, dependent on a definite velocity;

d, change of fluid ($a\,t^{-s}$); profoundly changes $(n-1)$;

e, possible change by reversal of direction of flow;

f, possible change due to stream and dissipation function.

Some additional remarks on some of these six factors are in order.

a.—Assuming Equation (26) to be correct, if it is applied to data where n is greater than 2, there results improbable values for ϕ. For example, in Table 4, the 22-in. wood stave pipe has $n = 2.215$ and $\phi = 0.013$; also, the Holyoke conduit, 103.38 in., $n = 2.08$ gives $\phi = 0.0129$. Judging by the remainder of the pipes, where n is from 1.74 to 1.95, these values of ϕ are probably about one-fourth to one-fifth of the real value. In general, both n and ϕ increase simultaneously. Here there is a decrease in ϕ with a great increase in n. For this reason, as well as that all the more exact experiments of Messrs. Saph and Schoder show values of n less than 2, it is believed that Statement *a*, is correct. It should be remembered that on the correctness of the statement $(n - 1)$ is 0 to 1 depends the nature of the function connecting the variables $(n - 1)$, ϕ V_0.

b.—This statement is in accord with general experience. The microscope measures of rugosity previously cited are also confirmatory. Although no great accuracy is claimed for such measurements, it will here be assumed that the following approximate relations exist, or sufficiently so for a comparison:

Seamless brass pipe, diameter of a rugosity 0.0035 cm., $n = 1.75$, $u = 0.975$.

Galvanized pipe, diameter of a rugosity 0.014 cm., $n = 1.92$, $u = 1.59$.

Where $(n - 1) = \tanh. u$, and taking logs.,

log. diameter	$\bar{3}.544$	log. u	1.989
" "	$\bar{2}.146$	" "	0.201
difference	0.602		0.212

From which there is sufficiently exact

$$6.5\,(\text{diameter of rugosity})^{\frac{1}{3}} = u \ldots\ldots\ldots\ldots (41)$$

$$\text{or } \frac{6.5}{N^{\frac{1}{6}}} = u \ldots\ldots\ldots\ldots (42)$$

where N denotes the number of rugosities per square centimeter. It is not affirmed that Equations (41) and (42) are correct. They are

only suggestive of some relation between the diameter of a rugosity and $(n - 1)$, which by Equation (42) would be

$$\tanh.\left(\frac{6.5}{N^{\frac{1}{6}}}\right) = (n - 1) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (43)$$

in which the 6.5 probably is a function of $(n - 1)$, V_0, and the mean height of a rugosity.

c.—As the critical points on a logarithmic diagram seem to be connected by straight lines, this seems sufficient to justify the statement that such lines are not asymptotes, but rather point to a condition of unstable equilibrium, which is the cause of the sudden change from one stage to another. Additional experimental work about the actual V_c points is needed to clear up the present uncertainties as to the precise cause of the change. Such work is also required to choose the proper form of function to connect the variables. The experiments indicate that V_0 and V_c are inverse functions of the absolute roughness. Concerning the suddenness of the change of stage, such a change may be considered as the integration of all the separate resistances caused by an average rugosity, and it seems probable that the more regular the surface the more sudden the change. Great irregularity might tend to changes before and after the V_0 or V_c point, with the result of a slight curvature connecting the branches; hence, experiments with burnished pipes are desirable to test this view and at the same time find some minimum value of n.

d.—Experiments with different fluids, especially with those having molecular constants which are best known, offer the opportunity to vary the exponent while using a given surface, which would doubtless throw much light on the problem. Conversely, the method would be applicable in chemical physics to clear up doubtful points as to molecular properties.

e.—This may be viewed as a special case of b where the arrangement of the rugosities differs for direct and reverse motion.

f.—From a study of the relations of $(n - 1)$, V_0, and ϕ, it seems probable that some additional function is present and that ϕ of Equation (26) is not strictly a roughness function but probably is of compound nature, hence the introduction of the stream function and dissipation function as possible factors. Such factors would certainly be functions of the roughness.

Summary.—The foregoing shows that there is one primary factor, the roughness, with V_0, V_c, and $(n - 1)$, dependent functions, and probably the stream and dissipation functions also dependent functions. Hence, the solution of the problem resolves, as a first step, into the formulation or definition of a roughness, and the assignment of a numerical value therefor.

Assuming temporarily, that ϕ is such numerical value of the roughness factor, and make λ a stream-line factor, it appears probable that the problem would be about as follows:

$$u = a\ \phi\ V_0\ \lambda\ (a\ t)^{-z} \qquad (44)$$

and

$$\tanh.\ u = (n - 1) \qquad (45)$$

For water, the term, $(a\ t)^{-z}$, would be constant, and could be dropped by giving a new value to a. The experimental work required to find the value of ϕ would simultaneously give V_0 and $(n - 1)$. λ, if required, would probably have to be deduced by mathematical methods, as it is undoubtedly dependent on ϕ and V_0.

Suggested Experiments.—Among the prime requisites for the final solution of the values of $(n - 1)$, V_0, and ϕ, might be named:

The use of clean or filtered and sterilized fluid. This would prevent changes of the surface condition and give a higher degree of accuracy to the constants thus found. Experiments could be repeated with considerable time interval if found necessary, with the certainty that the surface is unchanged.

Jacketing the pipe to avoid temperature changes.

The use of small pipes of at least two different diameters.

The use of burnished pipes, where the utmost degree of smoothness would be attained.

The use of threaded pipe, lead pipes, for example, having screwed surfaces. This would ensure a uniform condition of wall surface. The threads could be varied to any extent in pitch and cross-section of thread, and could have the thread of unequal section for direct and reverse flow. This method admits of great variations, is easily carried out, and would directly relate a given surface to the sought variables by means of exactly known surface conditions. By the change of fluid in the pipes mentioned, new values would be found for the variables.

By the use of photo-micrographs of actual surfaces, combined with such other measurements as might be practicable. It is probable that some definite numerical coefficient could be assigned to such surface.

After such work, would follow appropriate mathematical treatment of the acquired data, which, if not leading directly to the solution of the problem, would indicate the nature of any required additional experiments.

The writer is aware that the belief is held by some that it is doubtful if any better methods than those now used will ever be found, because of the manifold variations of the surface condition. From this belief the writer vigorously dissents. The present degree of knowledge of such surfaces is practically zero. When all the factors of the surface condition are studied and classified, a new world of information will be available, and thus narrow down the range of exponent and coefficient, if not to exactness, at least sufficiently close for practical use.

With these suggestions, the writer submits the paper, to hydraulicians and those interested in the subject, for criticism and discussion, and trusts that some of the hydraulic laboratories will take up the remaining work outlined herein, and find a complete solution of the problem.

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TRANSACTIONS

Paper No. 1290

THE DIVERSION OF IRRIGATING WATER FROM ARIZONA STREAMS.*

By A. L. Harris, Assoc. M. Am. Soc. C. E.

Most of the irrigating water obtained from Arizona streams comes from the higher mountains. Very little rain falls in the lower and more level country, where cultivation is practicable. For ages the typical stream has gathered its load of silt, sand, and gravel in these mountains and forced it, by reason of strong slopes, through the canyons and down to the low country, where grades become less, and it is spread out evenly in the broad, gently sloping, cultivable valleys. Usually, such valleys show evidences of having been flowed by slackwater for periods long enough to deposit deep surface beds of fine silt. The overflow has eventually cut its way out through surrounding hills, and worn down the outlet below the valley level. A channel has then been cut across the valley floor in the alluvium, toward which channel the surface slopes and drains on each side. The irrigated valley is now like a great shallow dish, inclined a little, and with the bed of the stream running across it in the direction of its slope. By intercepting the river water with a dam where it enters the valley from the surrounding hills, it may often be diverted into two canals, one to the right and one to the left of the stream. Throughout a wide circuit around the edges of the dished valley the canals are located,

* Presented at the meeting of February 18th, 1914.

with gentle grades to insure the flow and with the water surface generally a few inches above the natural ground to enable the water to be taken out at intervals. Of course, it is then possible to draw water into ditches from one of these canals to irrigate any piece of land between it and the river, as there is always a slope in that direction. As the river seldom flows through the middle of the valley, the area of irrigable land under each main canal is not the same. Indeed, it often happens that practically all the irrigable land of a valley lies on one side of the river. The great Salt River Valley about Phœnix has about twice as much irrigable land on its north side as is found on the south, and the Agua Fria and Buckeye Districts, on the Agua Fria and Gila Rivers, respectively, each utilize practically only one side of the stream.

Were it not that the arid nature of the country still allows erosion phenomena to go on vigorously, the provision of works for diverting the irrigating water would be much simpler. For some years the writer has frequently been called on to design diversion works in connection with both United States Government and private irrigation projects in this region, and this paper deals with the principal features and conditions relative to this experience.

Conditions to be Met.

The natural conditions in the locality of diversion present a stream having a flow which varies between very wide limits, and subject to very sudden changes. The floods often carry large quantities of water in great rushes, bearing down with them all kinds of drift—on the surface, in suspension, and grinding along the bottom—wood, silt, sand, gravel, and boulders. The works will be subject to much heat and dryness, but no ice (except on the plateaus of northern Arizona). The range of surface temperature is 160° or more. It is difficult to find an all rock foundation, as the river bed generally consists of a deep canyon, with rocky sides, which has been nearly filled with drift.

For the requirements of irrigation, diversion works must be such that the flow of water shall not be interrupted during long periods in the growing season, which, by the way, in southern Arizona, is nearly 12 months in the year. Canals should be kept free from sand and gravel, but the fine silt carried in suspension is valued highly by the

farmer, for keeping up the strength of the soil, very little other fertilization being necessary. If the system includes a storage reservoir somewhere up stream, much of this valuable silt, unfortunately, is deposited in the bottom of the reservoir, where, for the most part, it is not only useless but in time becomes a serious problem. The capacity to divert and distribute promptly unusually large quantities of water in flood times is generally required. Surface drift must be kept out of the canals. The layout of the whole works must also be such that the banks of the canals, where near the river, will be protected from the washing of the waste waters when floods pass down the river.

The Diverting Dam.

For diverting water into the canals, some kind of dam is generally required, although in special cases a subterranean collector conduit is used. It is generally necessary to raise the elevation of the water considerably in order to place the canals above the reach of flood water and to raise the surface level in them as much as possible for serving the greatest area of irrigable land.

The original diverting dam for irrigation was merely an obstruction of brush, stone, and earth projecting into the stream (as a wing) or across it, and deflecting the water into a ditch at about the natural level of the stream bed. Any small rise in the river destroyed it, and it had to be rebuilt. The point where the river water entered the canal was generally chosen at a place naturally suited to withstand the wear of flood, as by cutting through a projecting rock ledge or boulder point. The most serious expense and loss with such a canal head was not that of rebuilding the brush and stone dam, but the loss of crops due to the interruption of irrigation. However, by the hard and persistent work of the pioneers, most of the irrigated districts in Arizona were developed from such a beginning.

The next step in improvement was by the use of timber and rock-filled crib dams. Although a dam of this type can doubtless be made to perform the required service, its common history in Arizona has been that, because of insufficient protection of the foundations, or by ill chosen dimensions governing discharge, it has been sooner or later breached by a flood, causing heavy losses. In the absence of accurate flood data, and even when such data are at hand, it is difficult for the designer to appreciate the destructive action to be provided against

in damming these streams, except after first-hand experience. Usually, the river presents a very shrunken appearance, because of dry weather, and the intervals between extreme floods, often extending through a considerable number of years, give time to cover up and age the evidences of their action.

In this climate, timber is an even more temporary material than usual. All its possibilities for warping and checking are brought out by wetting followed by much heat and dryness. The necessary iron fastenings are hastened in rusting by alkalis often contained in the water. As this timber must usually come from Oregon and Washington, by way of Southern California, it is also expensive. At the present and prospective prices of cement, the expense of timber-crib dams in this locality is not justified by their enduring qualities.

Some diversion dams have been built of cemented stone masonry, but of late the most suitable and available material has proved to be concrete. Concrete materials are found at or near almost any diversion site. The excavated materials from the foundation trenches often prove to be quite suitable. Thus it appears that, for most cases in this locality, a concrete dam and intake works suitably designed and constructed is, both economically and structurally, about the ideal structure for diverting irrigating water. The concrete dam is usually carried entirely across the stream, but, on account of expense and poor foundation conditions, it sometimes appears advisable to build, next to the intake gates, a wing-dam of permanent construction, which acts as a spillway for moderate floods, and to maintain a dike of earth with a higher crest to a closure on the opposite river bank. This dike is expected to go out in the heavier floods, and then must be replaced.

The main features of the complete diverting dam, which is practically always of the overflow type, are the foundation, the apron, the rollway, the sand sluices, and the canal intakes.

Where bed-rock cannot be secured for foundations, excavation will generally discover beds of heavy boulders deposited by the stream, tightly packed together and chinked in with gravel and sand. On account of the usual position of the diverting dam at the mouth of a canyon, as mentioned previously, the heavier products of erosion coming from the mountains can be confidently looked for in the bed of the river. The deposits grow progressively coarser as the depth increases. A series of rather heavy walls is laid out and built in

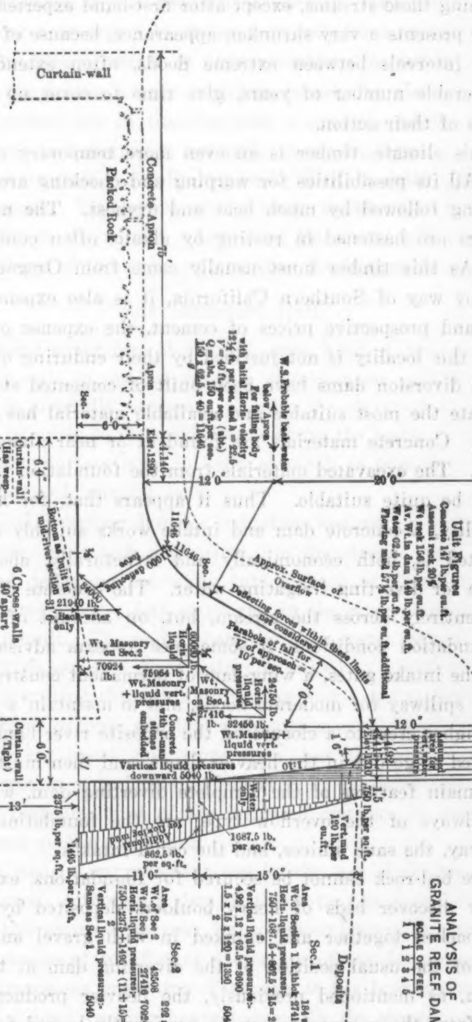
ANALYSIS OF
GRANITE REEF DAM
SCALE OF FEET
Curtain-wall
Concrete Apron
Packed Rock
Apron
Sec. 2
Sec. 1
Elev. 1290
W.S. Probable back-water below Apron
Unit Figures
Approx. Surface of Overflow
Deflecting forces within these limits not considered
Parabola of fall for V of approach = 10' per sec.
Wt. Masonry on Sec. 2
Wt. Masonry on Sec. 1
Wt. Masonry + liquid vert. pressures
Concrete with 1-man stones embedded
Vertical liquid pressures downward 5040 lb.
Curtain-wall (Has weep-holes in it)
Curtain-wall (Tight)
Bottom as built in mid-river section
6' Cross-walls 40' apart
Assumed High Water Elev. 1329
Deposits
Sec. 1
Sec. 2

FIG. 1.

trenches excavated down to the best gravel or boulder bed within reach. The dam is supported mostly on these walls, but is assisted by the undisturbed masses of gravel lying between them. The arrangement of walls consists of a deep one, put in by timbering and pumping, running lengthwise of the dam under the heel, and a somewhat shallower one under the toe. These two main walls are connected by cross-walls at intervals of from 20 to 40 ft. The heel-wall should be tight; the toe-wall is pierced by ample weep-holes to relieve under-pressure. The driving of sheet-piling is often impracticable on account of boulders. The foundation walls are brought up to the level selected for the bottom of the main section, and the material in the spaces between them is leveled to correspond. The solid gravity section is then laid on the whole as a foundation. By thus placing the bearing surface of the foundations deep, the danger of undercutting is minimized.

The principles of analysis used by the writer in determining a practical cross-section, for two of these dams which have been built and well tested, are simple, and can be seen by reference to Fig. 1, which was the writer's study in planning the Granite Reef Dam. The space behind the dam may always be expected to fill to the crest with river deposits, hence the provision for mud pressure. Although the cross-section is chosen for stability against overturning, using the bottom of the main section as a base, the foundation walls are monolithic with the upper parts, and contribute a heavy additional weight and frictional resistance acting against such overturning forces. As to sliding: the gridiron foundation walls enclose a very heavy mass of gravel which must be moved along with the structure or be separated from it by the parting of the concrete walls, either contingency being very strongly resisted. The underflow through the deposits beneath the dam has been found in excavations to be slow, and on account of the quantity of coarse material not easily washed, the first flood with its silt deposit makes an efficient stopper of seepage.

The upward pressure beneath the dam to be chosen for these cases is uncertain. The design of the Granite Reef Dam was made before the publication of the interesting analysis of upward pressures by G. E. P. Smith, Assoc. M. Am. Soc. C. E., and Professor H. C. Wolff, of the University of Wisconsin, contained in discussions on "Dams on Sand Foundations," by Arnold C. Koenig, Assoc. M. Am.

Soc. C. E.* Applying the test of that analysis, however, to Fig. 1, shows that the upward pressure was taken at more than enough to satisfy the requirements. By Mr. Smith's curves, on page 199 of that analysis, the average upward pressure on the whole bottom width of the dam, calculating H from the top of the apron instead of at the river bed, would be about 0.33 of the pressure due to a head of water of 20 ft. This would give a total of 18 000 lb. per running foot of dam on the bottom of the main section, as compared with 21 940 lb. used in the writer's old study; this, too, when applying the average pressure to the full width of bottom without excluding the 12 ft. taken up in the thickness of the foundation walls. Pressures on the bottoms of these foundation walls may be assumed to be greater per square foot than those on the base of the dam where they join it, but will not equal the weight of the walls themselves. All these upward pressures are lessened after the silting up has occurred above the dam.

The writer does not forget that the analysis, the test of which he applies here, assigns the maximum under-pressure at low water. He is also aware that the resultant of the upward pressure of Mr. Smith's diagram will not come in the middle of the base, as taken by the writer. However, the difference of head between the heel of the dam and the toe of the apron, for the case of flood assumed herein, will be about equal to the head at low water. The writer also finds that an upward pressure of 18 000 lb. applied to one-third of the base from the heel of the dam—as if the pressure diagram were a triangle—will still keep the total resultant force inside the middle third.

By the openings in the toe curtain-wall for relieving pressure, together with the method of jointing the apron, it is believed that the point of least upward pressure for the dam is at or near the toe of the main section, instead of farther under the apron. As will be seen later, though the pressure is relieved over the width of the apron, there is small chance for the surface or underground water to carry away material to cause undermining.

During a flood, as shown by Fig. 3, the water glides over the ogee and across the apron at a high velocity, and stands in a great wave just about at the down-stream end of the apron, where it strikes the gravel of the river bottom. The general level of the tail-water is several feet higher than that over the apron. At these times, so far

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* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 193.

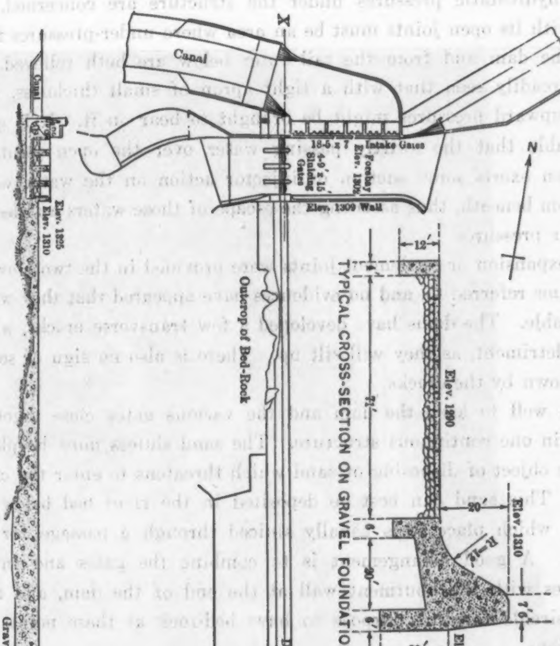
GENERAL LAYOUT OF GRANITE REEF DAM
Scale of Feet
TYPICAL CROSS-SECTION ON GRAVEL FOUNDATION
Canal
Intake Gates
Sluicing Gates
Elev. 1309 Wall
Outcrop of Bed-Rock
Dam
Apron
Direction of flow
Elev. 1290
Elev. 1310
Elev. 1292
Gravel
Canal

FIG. 2.

as the hydrostatic pressures under the structure are concerned, the apron with its open joints must be an area where under-pressures from above the dam and from the tail-water below are both relieved. It can be readily seen that with a tight apron of small thickness, dangerous upward pressures might be brought to bear on it. It is quite conceivable that the swiftly passing water over the open joints of the apron exerts some suction or injector action on the water which rises from beneath, thus assisting the escape of those waters and lessening their pressures.

No expansion or settlement joints were provided in the two Government dams referred to, and no evidences have appeared that they would be desirable. The dams have developed a few transverse cracks, which are no detriment, as they will silt up. There is also no sign of settlement shown by the cracks.

It is well to keep the dam and the various gates close together, usually in one continuous structure. The sand sluices must be placed with the object of disposing of sand which threatens to enter the canal intake. This sand can best be deposited in the river bed below the dam, to which place it is usually sluiced through a passage for the purpose. A good arrangement is to combine the gates and intake structures with the abutment wall at the end of the dam, and it is very desirable for this purpose to have bed-rock at these points for foundations.

Very careful attention must be paid to the apron laid along the bottom of the stream to protect the toe of the dam from being undermined. Even where a bed-rock foundation is practicable, the apron has been found to be a part of the structure very vital to its stability. Its surface should be placed at a level from 2 to 4 ft. below the natural bed of the stream. A gravel bar will then form just beyond its downstream edge which will back the water over the apron and keep a water cushion always at the toe of the dam for receiving the shock of the overflow. The width of apron sufficient to protect the toe of the dam varies according to the height and volume of the overflow and the consequent scouring energy of the water. The depth of high water above the crest will also influence the width. The apron is of concrete, but must not be connected to the dam, or in one continuous sheet. Being laid on beds of gravel and boulders near the surface, it will heave and settle more or less, and, if laid in a continuous sheet,

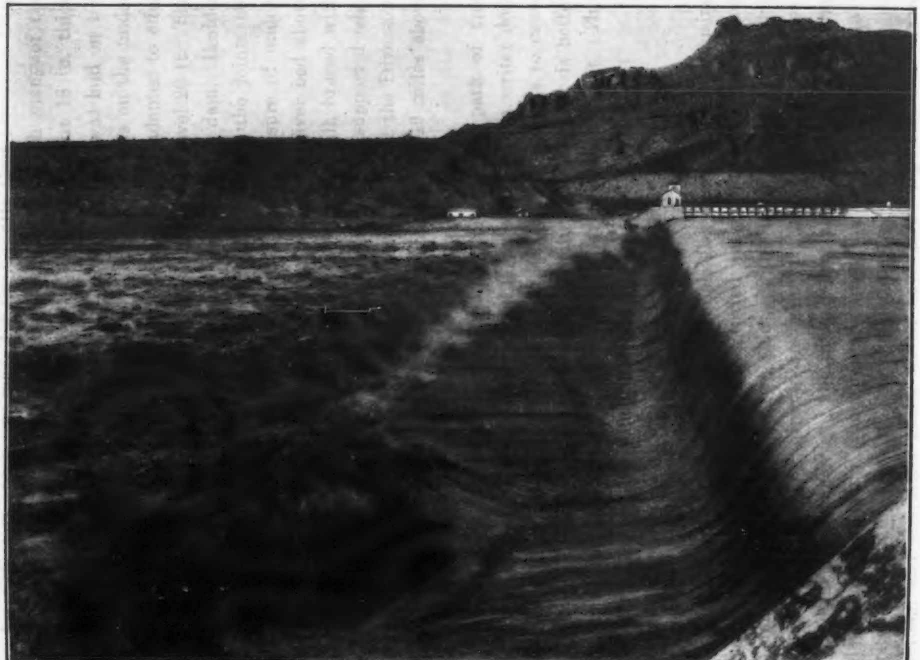

FIG. 3.—SALT RIVER PROJECT: GRANITE REEF DIVERSION DAM IN FLOOD.

FIG. 8.—SALT RIVER DISTRICT. GRANITE REEF DIVERSION DAM IN FLOOD.

this action cracks it into irregular, sharp-angled slabs which are likely to be shifted and undermined by the flowing water.

A satisfactory apron was made for the Granite Reef Diversion Dam by preparing a bed of packed clean boulders in which many of the stones projected prominently. A bed of concrete, about 18 in. thick and divided into 10-ft. square slabs by vertical joints, was laid on top of this foundation. On account of the adhering boulders on the under sides of the slabs, together with their weight, they are adapted to offer great resistance to sliding. The dam raises the water level 20 ft. The apron was 75 ft. wide, down stream from the toe to the dam. Besides dividing the apron into heavy slabs of definite shape, the joints between the slabs also served to relieve the upward pressure of underflowing water during the first few months, while the river bed above was sealing itself up with silt and sand. A curtain-wall, braced with piles and waling pieces, was used to strengthen the unsupported edge of the apron. A similar apron has proved satisfactory at the Diversion Dam of the Government Power Canal on Salt River, 19 miles above the Roosevelt Storage Dam.

The rollway has a surface sufficiently outside the path of free fall to be sure that an adhering nappe is preserved. The writer does not favor a turned-up toe for the main section, as it tends to cause increased agitation of the water on the main structure. It is better to let this energy be spent on the apron and on the bar below it. This result was secured at Granite Reef, as illustrated by Fig. 3.

Gates.

The entrance of sand into the canals has been resisted by using the principle of skimming the less turbid water from near the surface. A forebay is walled off on two sides, in the form of a comparatively deep channel running directly in front of the row of intake gates; it is open at the upper end and given outlet at the lower end by large sluice-gates emptying through an opening in the dam. The intake gates draw water from this forebay at a level considerably above its bottom, and the sluice-gates, which have large capacity, draw from the very bottom of the forebay, which has a good slope toward them. In entering the forebay, the motion of the water is checked by the increased cross-section, and the heavier sand grains settle to the bottom. At intervals the sluice-gates are opened suddenly by a hydraulic

piston, and a powerful rush of water washes out the sand. During a flood the water eddies about, and less sand is allowed to settle. With the gates placed in the usual way, if much water is drawn into the canals at these flood times, sand is carried in suspension with it. For this reason it is customary to build some kind of settling basin or sand-trap, within the first mile of the canal, where sand may be caught and sluiced back into the river. It would be better if all this separation of the sand could be done at the head-works. It is likely that the money expended on sand-traps or settling basins could, with equal advantage, be spent on the improvement of the intake sand-sluicing system.

To operate with good effect, the sand-sluicing gates must draw a current of water in front of every intake gate with enough velocity and agitation to scour thoroughly that part of the forebay. For this condition, care should be taken that the capacity to enter the forebay, as compared with the discharge of the sluice-gates, be not so large that the high velocity in sluicing occurs only close to the exit. On this account, sluicing for the benefit of the forebay cannot be done in flood time, although the sluice-gates are often left slightly open during a flood to prevent the banking of sand against them. As the water must always have enough room in entering the forebay to fill the maximum requirements of the canal, it follows that, to draw the level down and get a complete scouring at one operation, the sluice-gates must have a discharge greater than the canal requirements, unless indeed some device for controlling the entrance area to the forebay can be provided. Such a device would have to be on the outer wall of the forebay, in a very exposed position in flood season, and would add one more complication to the gate system. In the best examples now built there are rather heavy and expensive sluice-gates, on account of the large quantities of water to be handled. The writer has recently designed diversion works for a canal, to be taken out of the Gila River, in which he attempts to improve the sand-sluicing equipment. A modified arrangement based on that design is shown by Fig. 4 which he proposes as an improved type of intake works suitable for silt-laden rivers. In this design the skimming principle is carefully preserved for use in flood time when most needed. For the ordinary low-water conditions, the water is clearer, and is taken from the river over a series of wide, shallow crests, which reduce the

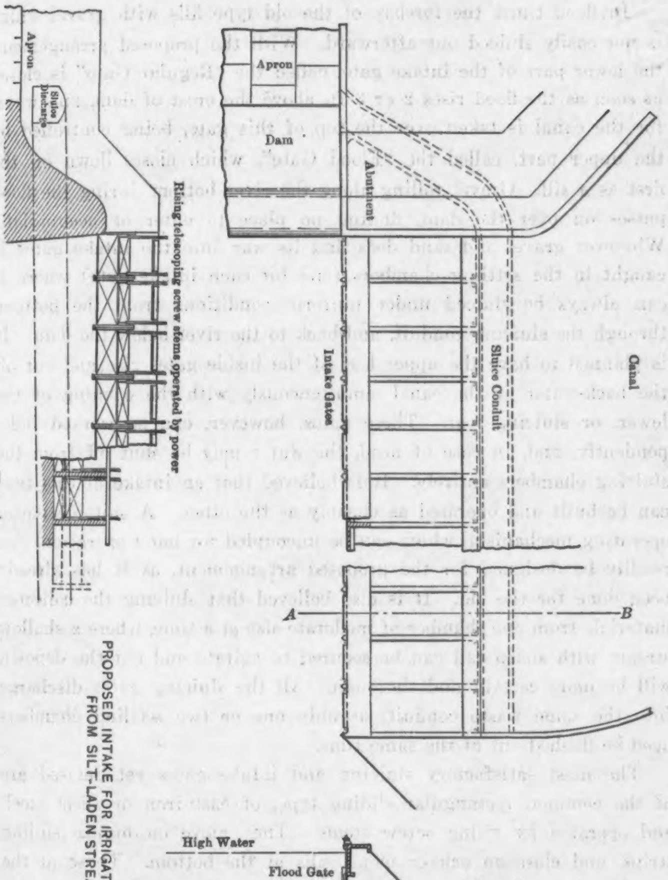

FIG. 4.

PROPOSED INTAKE FOR IRRIGATION WATER FROM SILT-LADEN STREAMS

CROSS-SECTION AT *A-B*

slope of the river bottom in the approaches to one so gentle that coarse materials will hardly be carried close to the gate entrances.

In flood times the forebay of the old type fills with gravel which is not easily sluiced out afterward. With the proposed arrangement, the lower part of the intake gate, called the "Regular Gate" is closed as soon as the flood rises 2 or 3 ft. above the crest of dam, and water for the canal is taken over the top of this gate, being controlled by the upper part, called the "Flood Gate", which closes down on the first as a sill. Gravel rolling along the river bottom during the flood passes on over the dam, finding no place to enter or accumulate. Whatever gravel and sand does find its way into the intake gates is caught in the settling chambers (one for each intake gate) where it can always be sluiced under uniform conditions from the bottom, through the sluicing conduit, and back to the river below the dam. It is planned to have the upper leaf of the inside gates rise and cut off the back-water of the canal simultaneously with the opening of the lower, or sluicing leaf. These gates, however, can be worked independently, and, in case of need, the water may be shut off from the sluicing chambers entirely. It is believed that an intake of this type can be built and operated as cheaply as the other. A suitable power operating mechanism, which can be uncoupled for hand operation, can readily be designed for the proposed arrangement, as it has already been done for the old. It is also believed that sluicing the collected materials from one chamber of moderate size at a time, where a shallow stream with sharp fall can be secured to agitate and cut the deposit, will be more certain and thorough. All the sluicing gates discharge into the same waste conduit, as only one or two settling chambers need be flushed out at the same time.

The most satisfactory sluicing and intake gates yet devised are of the common rectangular sliding type, of cast iron or sheet steel, and operated by rising screw stems. They move on bronze sliding strips, and close on oak or metal sills at the bottom. Those at the Granite Reef Dam, built by the U. S. Reclamation Service,* are of thin cast iron, of arched section on the compression side, the tension being taken by steel rods at the back and placed across the bow of the cast-iron shell. They are light, strong, and durable. In this case, the sluice-gates are operated by hydraulic pressure pumps, the

* *Engineering News*, January 7th, 1909.

transmission being by heavy chains running over sheaves; they are weighted with concrete for closing by gravity. At the Granite Reef Dam, a very excellent feature of all the gates, which were designed by F. Teichman, M. Am. Soc. C. E., is that, on the pressure side, where sand, etc., is likely to bank against them, they present smooth fronts, instead of a system of deep ribs, as often built in the past.

In designing gates for these situations, the question of the proper coefficient of starting friction to use for the bronze sliding ways comes up. The writer's experience has shown that this coefficient is much larger than published authorities known to him would indicate. In order to make a reliable determination of the coefficient under actual working conditions, the writer disconnected the raising mechanism of a 5 by 7-ft. cast-iron gate, having machine-bronze sliding ways, in the power canal at Roosevelt, and in its place attached a long timber lever resting on a fulcrum made of a piece of round steel shafting held between flat steel plates. A platform for carrying weights was then suspended from a definite point on the lever arm, and the apparatus was used to weigh the starting resistance. The actual weight of this particular gate was known from the inspector's weight obtained at the time of its receipt from the manufacturer. The timber lever was weighed and its weight per linear foot assumed to be constant. A correction for the weights of the lever arms was then made.

Statement of Conditions of Test.—

Water pressure on only one side of gate.

Size of gate opening..........................	5 by 7 ft.
Area subjected to water pressure on the gate, measured on the center line of closing strips.	38.7 sq. ft.
Depth of water at sill of gate................	8 ft. 8 in.
Head on center of pressure area...............	6 ft. 0 in.
Total water pressure on gate..................	14 500 lb.
Weight of gate................................	4 300 lb.
Sliding strips................................	Machined bronze.

Two experiments were made, as follows:

First Experiment.—To determine coefficient of starting friction after the gate had been closed tight for several weeks:

Result: Frictional resistance....................	9 062 lb.
Coefficient of starting friction.................	0.625

Second Experiment.—To determine coefficient of starting friction with the gate raised off the sill about ½ in. and water escaping under its lower edge:

Result: Frictional resistance........................ 8 985 lb.
Coefficient of starting friction................. 0.62

The turbid condition of the water, perhaps, is the chief cause of the increased size of the coefficient.

In nearly all cases it would probably pay to use gasoline engine power for operating the gates. These engines are now so common that one suitable for the power required may always be found, and speed in opening and closing is essential. They should be designed in such a way, however, that hand power may be applied in case of necessity.

Trash-racks and booms, for protecting the gates from driftwood, have not been found necessary. Most of the driftwood comes down in flood time, and then it is nearly always carried in the middle of the stream and over the dam, as it collects in the stronger current in that part of the river. If the dam is placed below, or on, a sharp bend in the stream (which is not good policy), the drift, of course, will be thrown close to the outside shore and will need to be guarded against.

Special Cases.

In Arizona there is much more land fit for irrigation than can ever be properly irrigated from the flow of the streams of that State, and because of this abundance of land, as compared with the available water in the regular flow, it is always in order to look for storage opportunities along a stream. It sometimes happens that a natural storage basin may be found near enough to the head of the farming area to make the storage dam serve the purposes of a diversion dam as well. Diversion is then a question of conduits and gates for conducting the water past the dam and into the canals. As these conduits and gates are a necessary part of the storage dam in any case, they need not be noticed in detail in this paper.

Occasionally, water has been diverted from sandy-bottomed streams of steep grade by collecting it in perforated pipes or timbered conduits buried in the stream bed and brought to the surface by extending at a gentle grade far enough down stream to gain the necessary elevation.

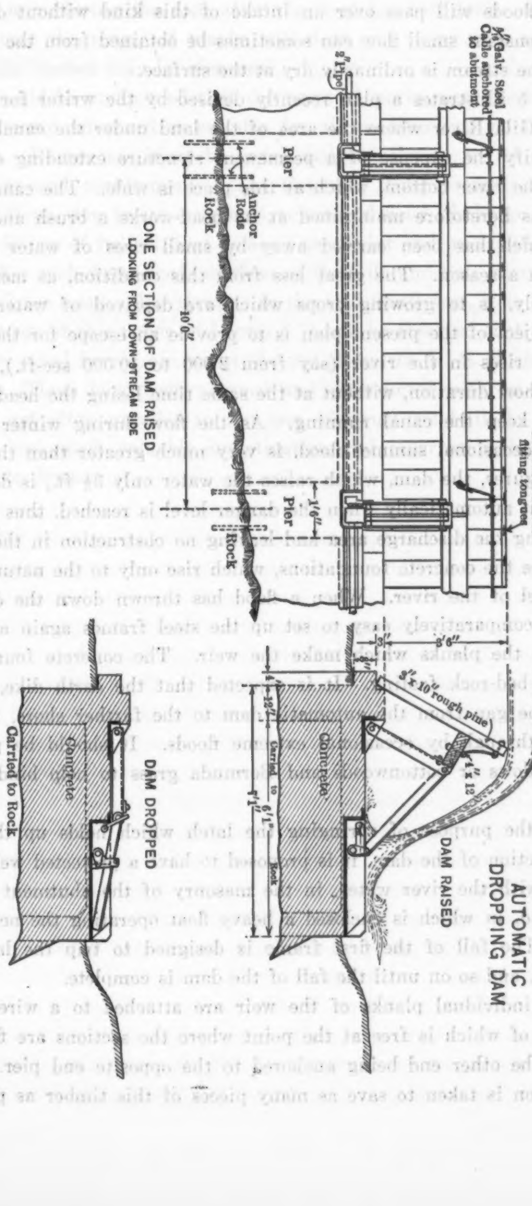
AUTOMATIC DROPPING DAM
DAM RAISED
DAM DROPPED
ONE SECTION OF DAM RAISED
LOOKING FROM DOWN-STREAM SIDE
Cable anchored to abutment
Grooves and loose fitting tongues
Pier
Anchor Rods
Rock
10'0"
1'6"
3'6"
3"
rough pine
Concrete
Carried to Rock

FIG. 5.

Heavy floods will pass over an intake of this kind without damage, and a constant small flow can sometimes be obtained from the ground where the stream is ordinarily dry at the surface.

Fig. 5 illustrates a plan recently devised by the writer for a case on the Gila River where the area of the land under the canal would not justify the expense of a permanent structure extending entirely across the river bottom, which at this place is wide. The canal company has heretofore maintained at its head-works a brush and earth dam which has been carried away by small rises of water several times in a season. The great loss from this condition, as mentioned previously, is to growing crops which are deprived of water. The main object of the present plan is to provide an escape for the small summer rises in the river (say from 2 000 to 10 000 sec-ft.), which are of short duration, without at the same time losing the head necessary to keep the canal running. As the flow during winter floods, and an occasional summer flood, is very much greater than the foregoing figures, the dam, which raises the water only 3½ ft., is designed to go out automatically when the danger level is reached, thus largely increasing the discharge area and leaving no obstruction in the channel above the concrete foundations, which rise only to the natural bottom level of the river. When a flood has thrown down the dam, it will be comparatively easy to set up the steel frames again and put in place the planks which make the weir. The concrete foundation is on a bed-rock footing. It is expected that the earth dike, which closes the gap from the automatic dam to the farther shore, will be broken through by occasional extreme floods. It should be planted with willows or cottonwoods and Bermuda grass to help bind it together.

For the purpose of springing the latch which holds up the first 10-ft. section of the dam, it is proposed to have a protected well, connected with the river water, in the masonry of the abutment at the shore end, in which is enclosed a heavy float operating the necessary lever. The fall of the first frame is designed to trip the latch of the next, and so on until the fall of the dam is complete.

The individual planks of the weir are attached to a wire cable, the end of which is free at the point where the sections are first released, the other end being anchored to the opposite end pier. This precaution is taken to save as many pieces of this timber as possible

for use in replacing the dam after the flood has run down. It is expected that the string of planks will float and swing over to shore, where the lumber may be recovered.

It should be understood that the types of works mentioned in this paper are not advanced as suitable for all conditions to be found in Arizona. Every case must be considered in relation to its peculiar conditions and requirements. The Colorado River presents a series of conditions, due to its enormous volume of flood discharge, and to the conditions of its bed, that demand particular and unclassified solutions. For such water-sheds as those of the Gila and Salt Rivers, however, and for many other streams in the arid Southwest, it is believed the contents of this paper will apply. It will be observed that the best examples noted herein are structures put in by the U. S. Reclamation Service. These have been planned and constructed with an amount of care not often equalled in the case of structures built by private parties, and much valuable investigation and observation have been devoted to them.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

Paper No. 1300

PAINTING STRUCTURAL STEEL: THE PRESENT SITUATION.*

By A. H. Sabin, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Allerton S. Cushman, Samuel Tobias Wagner, H. A. Gardner, A. W. Carpenter, Maximilian Toch, R. D. Coombs, Lewis D. Rights, W. E. Belcher, and A. H. Sabin.

The corrosion of structural metal (principally steel) by atmospheric and other natural causes is a subject which has long been of importance to the engineer. A few years ago, the greatly increased use of concrete structures aroused the hope that danger from such corrosion would be much reduced; but unarmored steel seems to be used as much as ever. Concrete has not taken its place, but has made an entirely distinct place for itself. Much has been written, and much has been done, relative to the protection of steel; but improvement has been slow, progress being made step by step.

The writer can remember when corrugated iron was introduced, some thirty years ago, that it was common practice to send out with a shipment a suitable quantity of powdered iron oxide to be used as a pigment, with directions for mixing it with oil and applying it as a paint. It was supposed that, because it contained iron, it was a proper paint to apply to iron, like "applying the hair of the dog to cure the bite" (which the writer has also seen done).

Some years ago, Mr. G. W. Thompson attempted to classify pigments, as to their relation with iron, by suspending them in water and immersing pieces of iron or steel in these mixtures. The results

* Presented at the meeting of January 7th, 1914.

were somewhat surprising; some of the pigments which common experience approved, seemed to increase corrosion in this condition, and others, known to be useless in protective paints, seemed to be much better for preventing it. Lampblack, for instance, was the worst in provoking corrosion, and white zinc or pulverized chalk prevented it. This was probably due to the fact that lampblack contains, condensed on the surface of its particles, considerable carbonic acid which is the most generally active agent in the corrosion of iron, and white zinc and chalk are basic substances by which iron is not rusted; however, the carbonic acid in lampblack is displaced by grinding in oil, and the well-known lack of durability in paints made of white zinc and chalk prevents their good qualities from coming into action.

So great is the need of more knowledge as to the value of pigments in paints, and their mode of action, that nothing promising new information is neglected. A committee of five chemists from different parts of the United States, with the approval of the Society for Testing Materials, made a series of tests of the principal pigments, and of some other substances, on steel immersed in water; and, as was to be expected, arrived at substantially concordant results. These results, as has been stated, were of no value from the standpoint of the paint-maker, being inconsistent with the known value of the pigments when ground in oil or varnish. When the report was published, however, the pigments were classified, according to their water value, into three groups, namely, inhibitors, indeterminates, and stimulators. This was the origin of the use of these now well-known words in paint terminology. It was expressly stated in the report that this was a classification as regards water only; but the names were so convenient and so tempting that those not familiar with the subject, and also many who saw their value for advertising purposes (two quite distinct classes), put them into common use to classify pigments in oil. It is obvious that any classification of pigments in oil should be based on their behavior in oil, and if, as must be conceded, this is radically different from water tests, the latter should not be regarded. All this investigation began some years ago; meanwhile, numerous young men, mostly students working under the supervision of their teachers, have made brief and generally inconclusive studies of paints, and almost without exception have used these indefinite terms, inhibitors and stimulators. Patents have even been

taken out—which, in the writer's opinion, are not only worthless but invalid—covering the use of old and well-known pigments. What is worse, every maker of a paint nostrum assures his hearers or readers that his particular paint absolutely inhibits rust, and that everything else stimulates it. This is the whole history of this jargon about inhibition and stimulation; it never had any particular value to the consumer, and it is generally used to mislead him.

It is obvious that in a good paint the pigment particles are enveloped in a film of oil; they do not come in contact with the iron; if they did, the paint would peel off, for no dry pigment adheres well to metal. It is as true to-day as it has been in the past that steel rusts because air and moisture act on it; and paints are used to keep air and moisture from it. They do not inhibit rusting, except as they inhibit the cause of it.

The important practical question is whether paints have been or can be improved as to being non-porous and durable. This is essentially dependent on the relation between the pigment and the oil. As to the true nature of this relation, very little is known; but something is known about its visible manifestations. It is known, for instance, that 1 lb. of dry red lead mixed with ¼ lb. of oil makes a paint of ordinary consistency, and 1 lb. of dry lampblack requires at least 6 or 8 lb. of oil, say, thirty times as much, or making allowance for difference in density, six times as much, as the red lead. Similarly, 1 lb. of white zinc takes twice as much oil to make a paint as 1 lb. of white lead; and white lead takes nearly twice as much as red lead. These are things we know; but we have no idea why they are so. Again, red lead, which is an oxide of lead, makes an excellent paint for iron; oxide of iron is neither very good nor very bad; oxide of manganese is bad. Our knowledge of paints is as yet largely empirical; chemists dislike to admit this; for, like everybody else, they hate to confess that there is anything they do not know, and thus, when a new theory is offered, some of them make a great rejoicing over it without first finding out whether or not it agrees with the facts. Where we are gaining is in more general appreciation of the value of the proper application of paint, better preparation of surface, more confidence in good paint rightly used, and in the better preparation of paint materials. For instance, in the older books, and until about twenty years ago, we find analyses of red lead showing as

low as 55% of true red lead, with 45% of litharge. Red lead is made from litharge, and the presence of the latter is not a sign of adulteration, but of incomplete conversion. At the same time other samples showed as high as 80% of true red lead. As is well known, there was much difference of opinion in those days as to the value of red lead as a paint for iron; though most users liked it, some thought it poor stuff. It is now known that its value depends on the quantity of red lead it contains. Coarse red lead always contains litharge, because the litharge in the middle of a large particle is never oxidized. It was observed that the finer the red lead, the better it was, and so a demand arose which forced the manufacturers to make higher grades; now they are grinding their litharge to an impalpable powder before roasting it, with the result that 94% of true red lead has been on the market for some years. Then an unexpected fact was developed. The old red lead when mixed with oil would set in a day or so—often in a few hours—into a cement, just like plaster of Paris and water; this tendency made it work with difficulty and unevenly in application, and its coarseness gave it a tendency to run; but the new, or high grade, article is inactive to oil, and brushes out smoothly like a house-paint. This enables the painter to cover 50% more surface with the same quantity and still get a coating having a uniform thickness which gives more protection than the thin portions of the paint formerly used. This secures greater economy, even at a slightly greater cost per gallon; and this is an economy, not only in the cost of the paint, but in the labor, because the paint works more easily, and a man can cover more surface in a day; it also requires less skill, and therefore, a less highly paid man, to do good work. For the last year or two, red lead ground in pure linseed oil has been offered to the trade as a paste ready to be thinned with more oil; such a paste keeps for a year or more, or indefinitely as far as known, like white lead paste. Its use saves time and waste in mixing, and, being ground through a mill, the mixture is perfect, which is not the case with hand-mixing; and, as it avoids the presence of a dusty pigment, it is more sanitary.

The only serious objection to the use of such red lead is that it dries more slowly than the older kinds. This can be obviated, however, by the use of a little japan drier. There is a well-founded prejudice against the use of excessive quantities of drier in any paint; but it

should be remembered that red lead paint mixed in the (standard) proportion of 28 lb. of pigment to 1 gal. of oil, contains 20½ lb. of pigment per gallon of mixed paint. If this pigment contains 15% of litharge, it has 3 lb. of litharge per gallon. Now, ordinary, good, lead japan driers, or lead and manganese driers of approved quality, contain the equivalent of 1 lb. of litharge in about 3 gal. of drier; and 3 lb. of litharge will make 8 or 10 gal. of drier. To make 1 gal. of mixed 94% red lead paint dry requires only ⅓ pint of drier; the rest is excess. It is much safer to add the desired quantity of drier. It may be asked why the litharge in the 94% red lead is not more active; it is probably because, when the peroxidation of the lead has been carried so nearly to completion, the particles of litharge are enveloped so completely by a dense coating of true red lead that the oil does not reach them. This is obviously not the case with the commoner and less thoroughly oxidized pigment.

It has sometimes been suggested, by those not very familiar with the chemical questions involved, that the litharge is the essentially valuable part of the paint, and that the red lead is only an inert extender. This is not so. The whole history of the subject shows that the improvement in red lead for paint during the last twenty-five years has been made by reducing the litharge contained in it; litharge alone, or used with other pigments, has not been satisfactory, though orange mineral, which is red lead free from litharge, is most excellent, and would be used if its cost were not so great. Further progress will undoubtedly produce red lead with a lower percentage of the protoxide; in fact, the 94% red lead now in the market usually contains much more than 94% of true red lead.

Progress has also been made in our knowledge of linseed oil. Within a few months, the American Society for Testing Materials has adopted specifications for North American raw linseed oil, which is of better quality than that made from South American seed. These specifications are the result of a great deal of work by many of the best oil chemists, and it is now possible for any good analyst to tell whether or not an oil is pure and good. Methods of paint analysis are in general being standardized; and a vast amount of work is going on in Germany and England as well as in the United States, on the chemistry and nature of drying oils. At present linseed oil

has adulterants, but no substitute; China wood oil is a valuable drying oil, more valuable for some purposes than any other, but, as an oil for ordinary paints, it is used, as far as the writer knows, only to cover up the use of non-drying oils which must be regarded as adulterants. At present prices, it is not likely to be used even in this way. Fish oil is used to some extent, as it always has been, in paint for roofs and smokestacks; but one should not be disturbed by talk about the "newer paint oils", for, except China wood oil, there are none.

In closing, it may be well to mention that the Committee appointed by the American Society for Testing Materials has made a final report on the condition of the paints on the Havre de Grace Bridge; as is well known, this bridge was painted six years ago by a committee of that Society, which committee included several members of the American Society of Civil Engineers. This report describes three of the paints as excellent; two of these were straight red lead in oil, and the third was red lead, with about 30% of a pulverized silicate added, in oil, the red lead in one test being about 98% true red lead. Nine other paints, of varying composition, are reported as affording generally effective protection to the structure. As all these paints were carefully applied, it is fair to conclude that the durability of any good paint may be increased one-half, and probably doubled, by proper care in its use as compared with average practice. It is only by continually reiterating this fact that we shall ever secure the most elementary and fundamental requirement for the economical treatment of structural steel.

DISCUSSION

Mr. Cushman.

ALLERTON S. CUSHMAN, ASSOC. AM. SOC. C. E. (by letter).—Mr. Sabin's exposition of the present situation in regard to paint protection of steel would appear to have at least the merit of simplicity. In a few brief paragraphs he sums up all that he cares to consider of the researches, theories, and conclusions of contemporary investigators of corrosion and paint problems, and sets forth what he terms the "whole history" of this "jargon" about inhibition and stimulation, which he states never had any particular value to the consumer, and is generally used to mislead him.

It does not seem fair or just that such a statement should be allowed to pass unchallenged, in view of the great number of engineers and others who are responsible for the care of steel structures and believe in the value of the "inhibitive" quality of protective coatings.

Mr. Sabin's review entirely ignores the electrolytic theory of the mechanism of the reactions which lead to metallic corrosion, and brushes aside all its application to paint problems, as of little or no value. Ignorance, even in the purest sense of the word, of a mass of accumulated evidence and data, is not argument, and is likely to be considered on a par with King Canute's effort to order back the advancing tide. The electrolytic explanation of corrosion, on which the inhibitive theory of paint protection is based, is now accepted by a great number of investigators and authorities in America and abroad. As the writer, to the best of his knowledge and belief, is largely responsible for the "jargon" which Mr. Sabin claims has done so much to deceive consumers, he may perhaps be permitted to contribute a few paragraphs to Mr. Sabin's "whole history" of the subject under discussion. As far as the writer is aware, the first use of the words "inhibition" and "stimulation", as applied to corrosion and protection problems, occurred in his paper on "The Corrosion of Iron".* In that paper the electrolytic theory of corrosion was developed. A few excerpts from this paper will serve to illustrate the point made:

"It has long been known that rusting is inhibited and that polished iron will remain bright indefinitely in all sorts of alkaline solutions, provided they are sufficiently concentrated. This is also true of all solutions of salts of strong bases and weak acids which hydrolyze to an alkaline reaction. This fact has been eagerly seized upon by the adherents of the various theories which have been advanced, as it can be made to fit in more or less well with them all. Thus, alkalies absorb carbon dioxide, and therefore carbonic acid is prevented from carrying on its work of destruction. The added fact that fully saturated bicarbonate of soda also provides full protection to iron, even in fairly dilute solution, which would seem to be a stumbling block, has not shaken the faith of the devout believers in the carbonic acid theory.

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* *Proceedings*, Tenth Annual Meeting, Am. Soc. for Testing Materials, June, 1907, p. 211.

Mr. Cushman.

"Solutions of chromic acid and its soluble salts, such as the chromate and bichromate of potash, inhibit the rusting of iron immersed in them.

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"The writer has observed that if a rod or strip of bright iron or steel is immersed for a few hours in a strong (5 to 10 per cent.) solution of potassium bichromate, and is then removed and thoroughly washed, that a certain change has been produced on the surface of the metal. The surface may be thoroughly washed and wiped with a clean cloth without disturbing this new surface condition. No visible change has been effected, for the polished surfaces examined under the microscope appear to be untouched. If, however, the polished strips are immersed in water it will be found that rusting is inhibited. An ordinary untreated polished specimen of steel will show rusting in a few minutes, when immersed in the ordinary distilled water of the laboratory. Chromated specimens will stand immersion for varying lengths of time before rust appears. In some cases it is a matter of hours, in others of days or even weeks before the inhibiting effect is overcome.

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"If iron or steel is brought into contact with water, either pure or natural, iron goes into solution by replacing the hydrogen ions that are present, and if oxygen is present, the ferrous ions are oxidized with the formation of ferric hydroxide in the form of rust. Any substance which increases the concentration of the hydrogen ions in the water, such as carbonic, sulphuric, or other acids, will stimulate corrosion. On the other hand, any substance which decreases the concentration of the hydrogen ions, or in any manner prevents the interchange of the electrostatic charge between the metal and the hydrogen ion, will inhibit corrosion. It follows from this that the rusting of iron is primarily a hydroxylization rather than an oxidation, and the oxygen does not directly attack the surface of the metal in the wet way or at ordinary temperatures.

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"Substances which increase the concentration of hydrogen ions, such as acids and acid salts, stimulate corrosion, while substances which increase the concentration of hydroxyl ions inhibit it. Chromic acid and its salts inhibit corrosion by producing a polarizing or dampening effect which prevents the solution of iron and the separation of hydrogen."

These citations are sufficient to show the writer's responsibility in the application of the words "inhibition" and "stimulation" to the discussion of corrosion problems. In the same paper, the practical application of the use of chromic acid and its salts as an inhibiting agent was suggested and foreshadowed in the following words:

"From what has been shown in regard to the inhibitive action of the chromates it is not improbable, since such dilute solutions prevent electrolysis and corrosion, that the addition of small quantities

Mr. Cushman. of bichromate to boiler waters would be highly efficacious in preventing the rapid pitting which has caused so much trouble.

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"The experiment has been made of keeping iron and steel in dilute boiling solutions of bichromate, through which a current of air is bubbled, for protracted periods, and as long as the strength of the solution was equal to or above one or two pounds of bichromate to 1 000 gallons of water no rusting has ever taken place. There would, therefore, seem to be no reason why potassium bichromate should not come into use as a boiler protective. The application of the various inhibitors in the priming coats of paints and other protective coverings has already been to some extent made use of, and it would appear that slightly soluble chromates should be theoretically the best protectives for the first application to iron and steel surfaces."

At the 1908 meeting of the American Society for Testing Materials the writer presented a paper entitled "The Inhibitive Power of Certain Pigments on the Corrosion of Iron and Steel", and in that paper the whole subject of the inhibitive theory as applied to paint protection was developed further.

Following the presentation of the writer's 1907 paper, and in the discussion of a paper by Mr. L. S. Hughes, entitled "The Deleterious Ingredients in Paints", Mr. G. W. Thompson, who is quoted by Mr. Sabin, suggested the water pigment tests as described by Mr. Sabin, and described some experiments which he had already made on this subject. Mr. Thompson, in the course of his remarks, stated that the remarkable thing which appeared in his original experiments was that some of the pigments had inhibited the oxidation, one pigment having caused a loss to the steel test piece of 0.37%, showing, according to Mr. Thompson's own words, a large inhibition, whereas another pigment had caused a loss of 2%, a considerable increase, as compared with the blanks which were run simultaneously. Mr. Thompson summed up by stating that he wished to make the suggestion to Mr. Cushman, Mr. Walker, and others who were investigating the corrosion of iron, that they make experiments to discover the relative electrolytic activity of various pigments in relation to iron. This suggestion was eagerly accepted by the writer, and, as Mr. Sabin describes, a committee of five chemists, from different parts of the United States, made a series of tests of the principal pigments on steel immersed in water. The results obtained by these laboratory investigations were then placed on the practical scale of experimentation by the steel test fence erected at Atlantic City by the Paint Manufacturers' Association of the United States, the inspection of which from time to time was placed under the supervision of suitable committees of the American Society for Testing Materials. As a result of these tests, which have been carried on constantly since 1908 and have been reported on annually, the best protection has been developed by the basic chromate of lead known as

American vermilion. The pigments which received the five leading grade marks in these tests depend on the chromate principle of inhibition. A number of the other leading pigments depend on the basic principle of inhibition, including both the orange mineral and red leads, which have also ranked comparatively high in the tests. Mr. Cushman.

In spite of this, Mr. Sabin states:

"It is obvious that in a good paint the pigment particles are enveloped in a film of oil; they do not come in contact with the iron; if they did, the paint would peel off, for no dry pigment adheres well to metal. It is as true to-day as it has been in the past that steel rusts because air and moisture act on it; and paints are used to keep air and moisture from it. They do not inhibit rusting, except as they inhibit the cause of it."

This statement would appear to show that Mr. Sabin has not comprehended the manner in which inhibitive pigments tend to protect steel when mixed with oil and painted on the surface of the metal. No intimate contact of the pigment particle and the underlying surface is in fact necessarily called for, in order that the inhibitive action should be exerted. A simple experiment can be made to prove this, by any one interested in making it. If a slightly soluble chromate, such as zinc yellow or the precipitated chromate of calcium, is mixed and ground with linseed oil, painted on a steel surface, and allowed to harden in the air in the ordinary way, and if this painted steel specimen is then immersed in water, it will be found by the gradual yellowing of the water that a small part of the chromate pigment is capable of dissolving in the water and even of being leached out of the film. This simple experiment shows that when a chrome inhibitive paint is used for protecting iron and steel, the water of the atmosphere which condenses or rains on a surface and is absorbed by the linoxyn coating, impregnates itself to a slight degree with the chrome radical of the pigment. This solution wets the metallic surface and to some extent maintains it in a passive or non-rusting condition. Though it is true that an ideal paint film should succeed in keeping air and moisture from the underlying surface, the fact is that paint films do not exhibit ideal qualities in this respect, and all of them are more or less permeable to water and atmospheric gases.

It is well known that for many years red lead has been used more or less successfully as a prime coating material on iron and steel, and the writer has no criticism to make of properly manufactured red leads which are properly mixed with a good vehicle and properly applied to steel. Unquestionably, it must be admitted that these red lead prime coaters yield good protection. When, however, Mr. Sabin claims that nothing more can be done, and that there is no need of further investigation of paint protection problems, the writer must take issue with him, as it can easily be shown under test that, based on the in-

Mr. Cushman. hibitive explanation of protection, many pigments are capable of giving better protection than even the best red lead. As a matter of fact, if the use of red lead as a prime coating material has already solved the problem of protecting steel to the maximum degree, it may well be questioned why it is that so large a percentage of structural steel all over the world is suffering from rapid corrosion and deterioration. Most of the inhibitive prime coating materials must inevitably cost more than red lead, and where, as is generally the case, the first cost of application is a factor of the highest importance, red lead will probably continue to be generally used. It is the writer's belief, however, that where the first cost of application is considered in relation to the maintenance or upkeep cost of structural steel for a number of years, it is now possible, as the result of research and investigation, to design an inhibitive prime coating material which will protect steel more efficiently than the usual red lead application, and he sincerely hopes that engineers and other consumers will not remain satisfied that the last word has been said with respect to paint protection in Mr. Sabin's presentation of the present situation in regard to the painting of structural steel.

Mr. Wagner. SAMUEL TOBIAS WAGNER, M. AM. SOC. C. E. (by letter).—The question of the selection of a proper paint to be used on a steel structure is one of the greatest importance, and much progress has been made in the last generation toward obtaining purer materials and applying them in a more workmanlike manner. For a number of years the writer has been of the opinion that it was not fair to expect any single paint to give the best results under different conditions of climate and exposure, and that generally specific conditions could be ascertained to determine which kind should be used. Throughout this discussion there were two general features which were kept prominently in the foreground; First: the materials should have a high degree of purity; second, they should be very finely ground.

For the past 20 years the writer has looked on red lead in linseed oil as one of the best primers for structural steel, and, when mixed with some other equally pure materials, as the best final coats. The first large contract under which he used this material was in 1897 on some bridges, aggregating about 5 000 tons, on which the following specification was used:

"Red lead in oil shall be of a good bright color and very finely ground. The pigment shall contain at least 95% of red oxide of lead; no sample will be accepted that contains more than 2% of foreign matter that is vitrified, or that contains metallic lead; or that when mixed with linseed oil and drying japan without grinding, and applied in a good body to a vertical surface of iron will not dry without running or separating."

The steel on this contract was erected promptly after fabrication, and no special opportunity was given to examine the wearing qualities of the primer, but it was undoubtedly the fact that there was difficulty in filling the specification, and, when filled, the resultant paint was of such a character that it could not be applied smoothly, but either ran or was streaky. It always had to be freshly mixed in order to work it at all. The general results, however, were good. Mr. Wagner.

Within the past 5 years the following experience was obtained with about 27 000 tons of structural steel bridges divided into several contracts. Nearly all the work on each structure was fabricated at one time, and the conditions on the work were that only half could be erected at that time, the other half having to be stored. Linseed oil was the primer specified on the first lot of contracts and after storing over a single winter it was found necessary to clean and re-oil a very considerable quantity of the metal. The specifications on the remaining contracts were modified by requiring that the priming coat should consist of red lead of the same specification as given above. Experience showed that after storage of more than 2 years the metal was in good condition.

The red lead required at this time was not obtained without very considerable trouble. The samples first submitted to test showed 49.08 and 56.15% of red lead; they were rejected, and the paint-maker promised to do better. Three samples were then submitted, showing 77.66, 79.56, and 83.26%, respectively, and these were rejected. These rejections caused a storm of protest, and it was stated that it was impossible to obtain a higher percentage of red lead in a mixed paint. After waiting some time the general contractor changed his paint man, and the next two samples showed 96.48 and 96.68%, respectively. The next three samples showed 98.11, 98.41, and 98.63%, and it is possible that if more paint had been needed the figures would have been higher. The paint when applied to the steel had a good bright color, and had none of the streaky appearance of the red lead of 1897. The color of such a paint is noticeably different from a paint with 75% of red lead. The fact that such paint with 94% and more of true red lead, as prepared by recent methods, will not set has not been noticed by the writer, but this has been probably because he has not followed up all the details. It is an important matter, and the manufacturers are to be congratulated on effecting this improvement, as well as the fact that it has greater covering value.

The writer was one of the railroad engineers who was requested by the Committee of the American Society for Testing Materials to examine the paints applied on the Pennsylvania Railroad Bridge over the Susquehanna River at Havre de Grace, and can testify as to the good appearance of the paints which contained red lead in the finishing coats. It is also of common knowledge that in many cases, when all

Mr. Wagner. paint has disappeared from structural steel, it is possible still to distinguish the shop marks of red lead where that paint has been used.

Mr. Gardner. H. A. GARDNER,* ESQ.—In referring to the behavior of various pigment paints, Mr. Sabin states that "our knowledge of paints is as yet largely empirical." This may be true in so far as it concerns any attempt to promote the use of a lead pigment, which is highly unsuited for the protection of structural steel, but it is a matter of great pride to the paint industry of the United States to point out the developments in paint technology which have been recorded during the past 10 years. In probably no other industry have such rapid strides been made. The achievements of Committee D-1 of the American Society for Testing Materials are especially noteworthy. The studies and researches of its members are largely responsible for the present state of this industry, in which the manufacturing processes are now controlled by the application of scientific principles. This change has led to a general abandonment of the slap-dash, secret-process methods of mixing paint, which at one time existed.

Regardless of the progress that has been brought about through research and practical test, it is not uncommon to hear the phrase, "some thirty years ago", repeatedly used by the skeptic in referring to the old-time methods and materials of painting. To assert that there is but one paint or pigment that can be used for painting any and all kinds of structures, is like asserting that one brand of patent medicine should be used for the cure of any and all ills. In these modern days, it is the practice to design paints so that they will properly meet special requirements. In no other way can the best results be obtained.

Unfortunately, Mr. Sabin's statements regarding the Havre de Grace Bridge tests are incorrect, and therefore misleading. In referring to the excellent results obtained with red lead paints, he states that the red lead used was 98% pure. The analyses made by the Department of Agriculture, at Washington, show that the red lead paints which gave good results varied widely in their composition. No. 6 paint consisted of 65% of red lead, the remainder of the pigment being made up of silica and other pigments. No. 10 contained 88% of true red lead and 10% of litharge. No. 11 contained approximately 95% of true red lead, the remainder consisting of litharge. Moreover, these three paints (the prime coats) were all made with different grades of oil and in different quantities. The oil in No. 6 which was present to the extent of 45%, had an ash content of 10.6, an iodine number of 140, and an acid value of 30. The oil in No. 10 was present to the extent of 30%, had an ash of 10, an iodine value of 157, and an acid value of 22. The oil in No. 11 was present to the extent of 23%, had an ash of 2.5, an iodine value of 163, and an acid number of 14.

* Assistant Director, The Inst. of Industrial Research, Washington, D. C.

Paints 12, 14, and 16, which also gave excellent results in these tests when used at the normal spreading rate of 600 sq. ft. per gal., were composed of mixtures of red lead with such pigments as zinc oxide, China clay, gypsum, silica, asbestine, and carbon black. Mr. Gardner.

It is hard to understand how any one can draw conclusions, from such a test, as to the protective value of different single pigments. The Havre de Grace Bridge tests have served a most excellent purpose in determining the relative value of various commercial brands of metal paints. These tests, however, cannot be cited as having contributed any definite knowledge relating to the effect of single-pigment paints as accelerators or preventives of corrosion. In fact, these tests were not designed for that purpose.

The only authoritative paint tests, to determine the efficiency of single pigments as preservers of iron and steel, which have ever been made, are those by the American Society for Testing Materials, at Atlantic City, N. J., in 1908. In these tests, which were made on a very extensive scale, the so-called rust inhibitive pigments proved the most satisfactory and received the highest ratings at every official inspection. Such pigments were either of the basic or the chromate type. The slightly soluble chromate pigments, such as zinc chromate, as Cushman had predicted, gave marked protection to the metal plates. The highly basic pigments, such as basic sulphate of lead, also gave excellent protection to the iron plates, to which they were applied as oil paints. Some of the most startling results of the test were recorded on test panels Nos. 34 and 36, and Nos. 9 and 10. Test panels No. 36 were coated with neutral chromate of lead. This pigment is neutral and highly insoluble. It gave, therefore, only fair results. Test panels No. 34 were painted with basic chromate of lead. On account of the basic nature of this pigment (high litharge content), it gave marked protection to the metal sheets to which it was applied; in fact, this pigment was given a rating of practically 100% by every inspector at every annual inspection. At the end of 5 years' exposure to the sea air, this paint is still in perfect condition, and will probably give excellent protection to the iron plates for many years to come. Panels 9 were painted with that type of neutral red lead which Mr. Sabin claims to be a good protective paint. This neutral red lead paint at the end of 3 years' test gave very poor results on every plate to which it was applied. The surface whitened, chalked, and became covered with little rust spots. The neutral and inert nature of this red lead made it of no more value as a metal paint than ordinary clay. On plates No. 10 there was applied a red lead containing a considerable percentage of litharge. At the official inspection of the tests, this basic litharge red lead received a very high rating, and completely demonstrated its superiority over the neutral red leads.

If the engineer is to adopt red lead as a primer for structural steel,

Mr. Gardner. he should specify that the red lead used, if in dry form, should be mixed on the job in oil, or if in prepared mixed form, should be a red lead of inhibitive properties. In the speaker's opinion, it should contain not less than 80 or more than 90% of Pb_3O_4, the remainder being litharge. Most high-grade red lead will run between 82 and 90% of Pb_3O_4. The red lead used should be extremely fine; 99% should pass through a 200-mesh screen. It should contain not more than 0.5% of impurities of any type. Engineers who are responsible for the upkeep of steel bridges, railroad equipment, and similar structures on which red lead is used, are rapidly becoming convinced of the value of basic inhibitive red lead. Specifications issued by the Union Pacific and Southern Pacific Railroads, and the City of Pittsburgh, call for a pure red lead having a minimum of 85% of Pb_3O_4. Specifications of the Rock Island System, the Illinois Central Railroad, and the Yazoo and Mississippi Valley Railroad call for a minimum of 80% of Pb_3O_4. The Public Service Commission of the State of New York has also issued specifications for a red lead having a minimum of 85% of true red lead. This red lead will be mixed with iron oxide, silica, asbestine, and similar pigments, and made into prepared paints. These paints will be used on the new elevated and subway roads.

That thoroughly inhibitive pigments are necessary for the protection of steel cars is evident from the practical tests made at the Pullman shops in Chicago, where thousands of steel cars are produced annually. The following quotation from a paper on this subject, as prepared by the chief chemists of the Pullman Company, presented at a recent meeting of the Master Car and Locomotive Painters' Association is quite illuminating, in view of Mr. Sabin's statements regarding rust inhibitive paints:

"The pigment must be of such material that it will be inhibitive, or, in technical words, must be alkaline to a slight extent and positive electrolytically to the steel or preferably a combination of both those factors. The Pullman Company, through their laboratory, devised a very comprehensive and careful series of tests, taking into account all the above factors and many more of slightly less importance. All these tests have checked up remarkably well with outdoor exposure tests on steel panels."

As this reference indicates, rust inhibitive pigments of the basic type are being used widely for the protection of metal surfaces. For this purpose, there are many pigments which have proved quite as suitable as the best grade of red lead. Among these may be mentioned basic sulphate of lead (blue lead), zinc oxide, and the red and black forms of iron oxide. Prepared metal paints made from such pigments have proved highly efficient in test and in practice.

In summing up this discussion, the speaker wishes to impress on the engineer the necessity of using, as a priming paint for metal, one

Mr. Gardner.

which will contain thoroughly inhibitive pigments, either of the basic or chromate type. Manufacturers of lead products may find that they can economically work up their off-color, low-grade, white lead for producing neutral red lead of 98% tetroxide content. Such a product may possibly be used in the manufacture of rubber goods, glass, or pottery, but is entirely unsuited for application to steel. To recommend such an inert pigment for the protection of iron, shows either a total ignorance of the well-recognized principles of metal protection, or an absolute disregard for the principles of conservation. In the speaker's opinion, it is quite as criminal to recommend a worthless paint for use on the steelwork of a modern structure, as it would be to allow the use of structural steel which is physically imperfect. In either case the ultimate result would probably be the same.

Mr. Carpenter.

A. W. Carpenter, M. Am. Soc. C. E.—In his brief and well-written paper, Mr. Sabin has given a great deal of very interesting information, from his point of view, on some of the newer theories and practices regarding oil paints for the protection of ferrous metals. He has used the opportunity to deprecate certain theories he does not himself accept, and devotes a large part of the paper to red lead, the paint pigment in which he is especially interested. He states that "when a new theory is offered, some of them [referring to paint chemists] make a great rejoicing over it without first finding out whether or not it agrees with the facts."

In spite of the speaker's sincere respect for Mr. Sabin's knowledge and experience, he is tempted to suggest that he is indulging in a little rejoicing himself over this new red lead theory, without offering much proof as to agreement with the facts. He states (1) that it is now known that the value of red lead depends on the quantity of true red lead it contains; (2) that the finer the red lead, the better it is; and (3) he suggests that red lead with more than 6% of litharge, when mixed with oil, will set in a day or so. This is evidently offered in opposition to another new theory which has been advanced, which is, that a certain proportion of litharge, at least in the form existing in the red lead as a result of the process of manufacture, is a benefit to the material as a paint pigment. The only evidence Mr. Sabin offers in support of his claims is the reference to the tests on the Havre de Grace Bridge, and this is not very satisfactory, as he does not state the litharge content of the two red leads used straight, and there might be some question as to the influence of the pulverized silicate on the third red lead mentioned. On the other hand, the only evidence offered in support of the theory that litharge is a valuable element in red lead, as far as the speaker knows, is the Atlantic City tests, in which ferrous plates coated with paint made with a red lead containing a proportion (stated to be about 12%) of litharge, were better protected

Mr. Carpenter.

after 5 years' exposure than similar plates coated with paint made with orange mineral, which contained practically no litharge.

The speaker's principal object in discussing this paper is to suggest that perhaps both these contradictory theories are wrong, or, at least, not worth the fuss that is being made over them. This suggestion is the result of a series of tests of samples of red lead of varying composition, as shown in Table 1.

TABLE 1.—TESTS OF RED LEAD FOR NEW YORK CENTRAL LINES BRIDGE ENGINEERS' COMMITTEE.

No. of sample.	Chemical Analysis. True red lead. Percentage.	Chemical Analysis. Litharge. Percentage.	Chemical Analysis. Other elements. Approximate percentage.	Fineness. Relative fineness.	Fineness. Weight per cubic inch, in grammes.	Spreading Capacity of Paint. 24 lb. lead to 1 gal. oil. Square feet covered.	24 lb. lead to 1 gal. oil. Relative percentage.	24 lb. lead to 1 gal. oil. Working qualities.	30 lb. lead to 1 gal. oil. Square feet covered.	30 lb. lead to 1 gal. oil. Relative percentage.	30 lb. lead to 1 gal. oil. Working qualities.
1	94.2	5.7	0.2	2	27	23	74	O. K.	30	79	O. K.
2	92.9	7.0	0.01	3	33.6	20	65	Covered poorly and brushed out hard.	29.5	77	O. K.
3	87.7	11.7	0.5	1	23	28	90	O. K.	33.5	88	O. K.
4	84.5	15.2	0.3	3	33.6	31	100	Mixed thin and sagged.	38	100	O. K.
5	80.6	19.1	0.3	1	23	20	65	Brushed out well but ran.	27	71	O. K.

Quantities for spreading tests: ¼ pint of oil to 12 oz. and 15 oz. of lead, respectively.
Spreading tests made May 18th and 22d, 1911.

This table shows the results of the examination and tests of five samples of red lead, each furnished by a different manufacturer. It shows the fineness of division, the chemical composition, and the spreading capacity when mixed with linseed oil in the usual paint proportions. It will be noted that the true red lead content ranges from 80.6 to 94.2% and that of litharge from 5.7 to 19.1 per cent. The fineness determination was made by the weight method, and is merely a comparative one based on the theory that the fineness of division of materials of equal specific gravity and similar structure of particles will vary inversely as the weight of equal volumes of the materials when equally packed. The weights given were determined by sifting the leads through a 20-mesh wire screen, placed at a fixed height above the measure of volume until the latter was filled, striking the material off flush with the top of the measure, and then weighing the contents. The lead with the lowest true red lead content (Sample 5) was found to be a tie with one other as the finest in division of all five leads; and the lead having next to the lowest true red lead content (Sample 4) was a tie with one other, the one with next to the highest true red lead

content (Sample 2), as the coarsest. The other cases exhibit a similar lack of law of relation between true red lead content and fineness of division. Mr. Carpenter.

The paint mixtures for the spreading tests were made with open-kettle boiled linseed oil and no drier. They were applied to clean, galvanized, sheet-metal surfaces—well weathered and apparently all in the same condition—by one operator, on the same day, and under the same weather conditions, using a separate clean brush of one size and kind for each paint. The quantity of paint was small, but covered an appreciable area in each case, and the results with the two proportions of lead and oil will be noted as generally consistent. They show that one of the two coarsest leads (Sample 4), and the one having next to the lowest true red lead content, gave the greatest spreading; and one of the finest leads, which had the lowest true red lead content (Sample 5), gave the smallest spreading.

It seems safe to say that, if these tests are a correct indication, there is no definite relation between the fineness, the proportion of true red lead, and the spreading capacity of the pigment in oil, but that other elements control.

The foregoing determinations, of course, do not give any indication as to the durability of the respective leads in paint, nor their respective values for the protection of ferrous metals from corrosion. In the endeavor to determine these qualities, the leads were mixed with boiled linseed oil in the proportions of 20, 25, and 30 lb. of lead to 1 gal. of oil, and the resulting paint was applied, under three different conditions, in one coat each on steel plates 5 by 7 in. in lateral dimensions. The paint was prepared by first pouring a small proportion of the oil over the full quantity of red lead and letting it stand over night. Next morning, the remainder of the required quantity of oil was added, and the whole was stirred thoroughly.

Three lots of paint, one for each proportion of 20, 25, and 30 lb. per gal. of oil, were thus mixed from each of the five samples of red lead, thus making fifteen lots. From each of these fifteen lots a steel plate was coated and properly marked for identification. Then from each lot, two small wide-mouthed bottles were partly filled. One of each of these bottles was allowed to stand open, without stirring, for a period of 2 weeks, at the end of which time the paint was stirred and applied to a steel plate in a single coat; in no case had the mixture set so that it could not be readily stirred and applied in a good coat, although in one or two cases the coats were quite thick. The paint in the second set of bottles was stirred every day for 2 weeks and was then applied to steel plates, except that one lot had set so that it could not be applied; that one was made from Sample 5 in the proportion of 30 lb. of lead to 1 gal. of oil; this sample contained the greatest proportion of litharge, 19.1 per cent.

Mr. Carpenter. After coating these plates, they were exposed to the atmosphere on the roof of an office building in New York City and left for 463 days, or 1 year and some 3 months. They were then brought indoors and examined, with the following result:

The plates to which paint was applied immediately after preparation were in generally good condition. Those coated with 25 lb. to 1 gal. were in the best condition, although the coatings had changed color. There was no choice between the coatings with leads made by the different manufacturers. The 30-lb. per gallon coatings were too thick in some cases, and showed more breaks, with rust spots, than the 25-lb. per gallon coatings. The 20-lb. per gallon coatings showed the best color, probably due to the greater protection of the pigment by the oil.

The coatings with the mixtures which stood 2 weeks before application in many cases were as good and had protected the steel as well as those applied immediately after preparation; in some cases it was evident that the mixtures had commenced to thicken or "set" at the time of application.

The coatings applied after the mixtures had been stirred daily for 2 weeks gave good protection, with two or three exceptions. They generally were not quite as good as those from the paints which stood 2 weeks without stirring, and the latter were not quite so good as those from the paint applied immediately after preparation. In several cases the paint had begun to thicken up or "set" when applied, but the results from these coatings were decidedly good.

In view of the foregoing there does not seem to be anything to choose between the different leads, as affecting the durability and protective qualities of paint coatings in which they are incorporated. For economical reasons, the choice would seem to lie with the one that spread the farthest, but there does not seem to be any way to predetermine the spreading capacity of paint made with any particular lot of red lead, from the fineness of division or litharge content of the lead, at least within the range exhibited by the samples tested.

It may be thought that the exposure tests were not long enough for comparative results. In this regard attention is called to the fact that the tests were made on single coats, and that the time was rather long for such.

It is not the intention to offer these tests as in any way conclusive, but simply as some definite determinations, as opposed to mere statements of opinion. It is hoped that others may take up the investigation, to the end that the facts may be established. It would seem to be a good field for investigation by the National Bureau of Standards.

Mr. Sabin's paper would lead one to think that red lead is the only good pigment for paint for preserving ferrous metals. Although it

is undoubtedly one of the best of the practical pigments for use under many conditions, there are other conditions under which it is not satisfactory; and it is doubtful if its use in more than one coat is desirable on structural steel for ordinary exposure, owing to its cost, its susceptibility to deterioration in color and integrity in contact with gases, and its often objectionable color. Mr. Carpenter.

It may be very well argued that one or two under coatings of red lead paint will be generally improved by over coatings of paints with other and cheaper pigments, rather than additional coats of red lead. The speaker has also known of many instances in which oxides have made remarkable records for durability and protection in paint coatings applied directly to steel surfaces, and he has a much higher regard for the value of certain iron oxides for use in structural steel paints than the author expresses for this class in general.

MAXIMILIAN TOCH, ASSOC. AM. SOC. C. E.—This paper can hardly be said to throw any new light on an old subject; an analysis of all that it contains can be summed up as a propaganda for an alleged new material known as red lead ground in oil, and sold in paste form. Mr. Toch.

Mr. Sabin's conclusions with reference to the condition of the paints on the Havre de Grace Bridge and the final report are incomplete. The report really describes as excellent nine paints, three of which are red lead; and one of these is a ready-mixed red lead containing, as he says, 15% of a pulverized silicate. Now, to be frank, such an indefinite term as "pulverized silicate" should not be used. The paint in question contained a substance known as aluminum silicate, but better known to engineers as ordinary American clay. The statement that 15% of the composition of this paint was clay is also misleading. Mr. Sabin, however, has corrected this by stating that the percentage of clay is nearer 25. The specific gravity of clay is a little greater than 2; the specific gravity of red lead is about 9. The volume of these two materials is such that the actual quantity of clay in this paint was nearly as much as that of red lead. In other words, 25 lb. of a good, clean clay has the same bulk as 85 lb. of red lead.

Now, what has the author proved? That the two pure red leads in these three results were only as good as the red lead which was mixed with clay, and, therefore, that clay, when mixed with red lead in moderate proportions, is just as good as red lead, for, after 6 years, there has been no difference in these three test plates. Then again, every one of these red leads was coated over with a carbon paint, and, if the test proves anything, it proves that the carbon paint was a perfect excluder of oxygen and water, and therefore no rust resulted.

The speaker has examined the report of the Havre de Grace Bridge committee very thoroughly, has been active on this committee to some

Mr. Toch. extent, and has inspected these plates. Mr. Gardner has stated that among the paints which have turned out the best was that on Plate No. 12, and as nearly as can be determined from the report, that plate is the only one having two coats of paint, all the others having three coats; and the 600-sq. ft. spreading test is as good as the best. The speaker is not ashamed of this, in view of the fact that it was a paint invented by himself and made by his firm. Bearing out Mr. Gardner's statement with reference to the inhibitive quality of the basic paints, that on Plate No. 12, according to the published analyses, was the only one which was alkaline, due to the fact that the priming coat was a now fairly well-known cement paint. The finishing coat was a carbon paint, and after 6 years these two coats are equal to the three coats of the others.

Four smokestacks for the power-house of Long Island City were painted in August, 1906, and are in as good condition to-day as they were 7 years ago. In that case three coats were applied, but the first was painted directly on rust, and that rust has never progressed sufficiently to scale the succeeding coats of paint.

The author has not even "damned by faint praise" any other material as a paint. On December 13th, 1913, the speaker examined very critically the test plates at Atlantic City which were painted in 1908 with single pigments, where, for instance, oxide of iron, red lead, graphite, etc., were applied without any subsequent protective coat, and he must take issue with Mr. Sabin, for the three red lead plates which are still on exhibition there are not as good as the three plates covered with Prince's metallic brown, which is an oxide of iron; nor is the American orange mineral, which is a pure form of red lead, anywhere near as good as the sublimed blue lead. In fact, to be perfectly frank, the sublimed blue lead is far better than either red lead or orange mineral.

Allusion must be made to Mr. Sabin's statement: "one should not be disturbed by talk about 'newer paint oils'." The author did not mean "newer paint oils" when he put those words in quotation marks, but intended to refer to "newer paint materials," which was the subject of a lecture delivered by the speaker in London nearly 3 years ago, and the speaker is quite sure that Mr. Sabin is not so much against progress as this statement would indicate. Considerably more than 100 000 copies of that lecture were reprinted, and it has been quoted extensively both in Germany and in France, therefore, when Mr. Sabin pretends that there are no newer paint materials, the speaker is not inclined to take him seriously.

The speaker has always held the strong opinion that no man should take out of the common store any more than his allotted share. To him it is just as criminal to corner the wheat market and raise the price of bread as to take money out of the pockets of other people;

there is this difference, however, the pickpocket can be prosecuted, but the man who raises the price of bread can not. Mr. Toch.

About 4 years ago, at a time when the crop of turpentine was normal, some men in the South cornered the market. Hundreds of thousands of gallons were stored, and none of it was sold until the price, which should have been about 45 cents, was raised to $1.13 per gal. The speaker very quietly cast about for a substitute for turpentine, and so did many others. There was one man, in the employ of the United States Navy, who showed much more courage than any admiral ever did in battle, when he wrote a specification for turpentine substitute for the United States Navy, and in one year reduced the purchases of turpentine in the Navy about 70 000 gal. A large number of other progressives did the same thing, with the result that to-day we can do without turpentine absolutely, and the price in its tumble shattered those who had raised it abnormally.

About 3 or 4 years ago, through the failure of the flaxseed crop in the United States and other countries, the price of linseed oil rose from 36 cents to $1 per gal. Although the speaker has never contended that he had found an oil which was just as good as linseed, he has proved that there are several which, in conjunction with linseed oil, make most excellent paint. If the author thinks there is nothing in the newer paint materials, let him ask the manufacturers of printing ink, linoleum, artificial leather, and also a number of paint manufacturers, whether Soya bean oil and Menhaden oil are not valuable assets to them to-day. A few months ago one of the largest manufacturers of Menhaden oil stated that where he sold one barrel to the manufacturers of paint and linoleum 10 years ago, he is selling a thousand barrels to-day. In the speaker's lecture on fish oil he pointed out that many of the failures were caused by the ignorance of the men who made the tests. On any price list of fish oils to-day will be found whale oil, porpoise oil, and seal oil, under the heading of fish oils, and it was the speaker who pointed out to the paint industry that these are animal oils and not fish oils, and, further, that a fish oil, to be suitable for paint material, had to possess certain constants, otherwise it would be useless.

In 1896 5 gal. of China wood oil came to the United States. As far as known, it was the only China wood oil imported that year, and the speaker received those 5 gal. During the year ending October 31st, 1913, 5 400 000 gal. of China wood oil were consumed in the United States. This oil, before it was understood, had a very bad reputation as an adulterant and an undesirable paint material. When one talks about adulterants one is treading on very thin ice. Everybody is familiar with the high-priced Epicurean dish known as terrapin. To serve terrapin at $3 per microscopic portion is considered in high society a great feat, and yet, 150 years ago, when a good slave was

Mr. Toch. sold in Maryland, the agreement generally made was that the slave must not be fed on terrapin, for at that time it was considered hardly good enough food for dogs. If China clay, which is the pulverized silicate mentioned by the author, were equal in price to red lead, a feature would be made of its use instead of a secret.

This paper tends to show that there is a new material on the market known as red lead ground in paste form, which can be very easily thinned and made ready for use. Pure red lead, ground in pure linseed oil, in paste form, has been known for many years, and, what is more, the author knows it has been known, because orange mineral, which is a pure form of red lead, has been used by color grinders as far back as the speaker can remember. The speaker made ready-mixed red lead more than 20 years ago, and made it successfully; and it remained in its ready-to-use form. He has no quarrel with red lead, because he thinks it is a very good material. It is not better than sublimed blue lead, nor is it better than certain forms of oxide of iron. It is certainly very much dearer than any other paint, and, on account of its great weight, it is harder to apply than a lighter one; it does not stop progressive oxidation, nor is it a fit material for use on steel which is to be bedded in concrete, not because of the red lead, but because of the oil, which is readily saponified.

Everybody must admit that in some places red lead is a most excellent protective paint, but in other places it does not give such a good account of itself. Then again, every paint chemist knows that there are some red leads that contain nearly 1% of caustic soda, which is quite sufficient to destroy a paint when it comes in contact with water, and all know that the red lead which can be ground in paste form and remain plastic is not the red lead that has been used heretofore, which contains litharge, and the old litharge red lead paint should not now be condemned because it forms a cementitious material.

As a paint under water, red lead is sometimes good and sometimes bad, depending, of course, on whether or not it has been made by the nitrite of soda process.

A large quantity of red lead was used on the steel of the locks of the Panama Canal as a priming paint, three coats being applied in many places. When submerged in the lake water, the red lead turned blue, owing to the action of the sulphur gases on the lead, and the paint was in many places entirely destroyed. Whether this was due to the fact that it contained a small percentage of free caustic soda and thereby became soluble in water, or whether the decaying organic matter destroyed it, amounts to the same thing. Preliminary tests, however, may have demonstrated these defects.

The speaker has at all times stood for fairness and the exact representation of facts with regard to paint materials. He has spoken well of all those materials which his judgment and experience have taught

him were good. He has even had the temerity to talk well of his own paints whenever he has had the chance, for the world is large enough to support all, and to give every one a share. He is glad of the opportunity to state that he and Mr. Sabin have been friends for many years, and he respects him highly. In differing in opinion with him, he is simply breaking a lance in the interest of paint science. The author is a man who is well equipped, and has had a long and varied experience which entitles him to the right to talk, but the speaker agrees with the poet Goethe, *"Der Mensch verhöent was er nicht versteht"*, which, translated into good "American", means "A man is down on a thing when he is not up on it". Mr. Toch.

R. D. COOMBS, M. AM. SOC. C. E.—The manner in which paint is ordinarily applied in commercial practice is so irregular and faulty as to render any direct comparison inaccurate. The painting, as usually done on structural steelwork, is about the poorest job on the entire construction. As most erecting engineers know, the work of painting is disliked by the higher priced men—the iron-workers. Instead of having the work done by a $4 or $5 iron-worker, it is done by a $2 to $3 steel painter or apprentice, and it is frequently a very poor sample of painting. Again, in the entire painting, the shop coat would seem to be the thing which can most easily produce a poor result. Mr. Coombs.

The average paint used in the shop is a relatively cheap material, costing about one-half as much as a good field coat, and it is frequently put on by the cheapest labor in the shop, and more frequently it is put on very badly. Now, it would appear reasonable to expect that a poor shop coat over an imperfectly cleaned surface would have a very great bearing on the final net result of the work. Thus, if a $0.60 paint is put on in the shop as badly as is frequently the case, and then a $1.25 paint is used in the field, it seems open to question how much of the result of the final painting is chargeable to the shop coat and how much to the field coat.

Referring to the matter of red lead paint as practically applied—not as a laboratory test, but as done in the field—the question arises, how many pounds of red lead to a gallon of oil should one expect low-grade painters to use successfully. There are in existence a number of specifications, issued by individuals and corporations of high standing, calling for 33 lb. of lead to 1 gal. of oil. On a rather important piece of construction, in the vicinity of New York City, the master painter, who was in the employ of the company and had no object whatever in cheapening the work, reported that he could not put on as much, unless he was allowed to use a trowel. This may have been an exaggeration, but it is the speaker's recollection that about 22 lb. was as much as the men found they could work with to advantage.

The speaker does not wish to convey the impression that the painting in the field is a much better piece of work that that done in the shop,

Mr. Coombs. but that the painting, either in the shop or in the field, as usually done, is not a good job. Further, the manufacturer alone is not to blame, but also the owner and the engineer. It is simply a bad practice which has become very common, and for which all concerned are at fault. Reference is not made particularly to red lead paint, but to all paints.

There seems to be some tendency on the part of the manufacturer to regard the multitude of paints now specified as a senseless nuisance, and—it must be admitted—with perhaps some justification, from his standpoint. However, having recovered from his chagrin at the receipt of an order embodying another change in paint from the kinds in immediate use in the shop, and after faithfully incorporating the proper instructions in the shop orders, he often takes no further interest in the matter. The paint question is now up to the shop foreman, whose theories thereon are often in need of expurgation as well as elaboration. To the minds of many shopmen, who take a real pride in doing good shop work, any old paint of the specified color is good enough.

Turning now to the paint man, it is difficult to understand how he can escape responsibility for at least part of the skepticism of the manufacturer, engineer, and owner, as many paint salesmen rather permit the impression to get abroad that the paint used by their competitors will eat the steel rather than protect it.

The private owner is not so much to blame, as he is neither an engineer nor a paint expert. Those corporate owners, however, having engineering departments, whose duty it is to conserve their construction interests, can hardly be absolved from blame for careless work.

The speaker is unwilling, although able, to specify definite instances in which a cheaper grade of paint than that specified has been used, or in which either, or both, the shop and the field painting have been very badly done. Painting in wet or freezing weather, the use of liberal quantities of "thinners" and benzine, painting over scale, rust, cinders, mud, etc., and the omission of paint, are not such rare occurrences as optimists would have us believe.

It would seem that it is about time for a reformation in the methods used in painting. Not so much in relation to the work done under efficient inspection as in the great bulk of work included in small contracts throughout the country, which are either poorly inspected or not at all. Where paint is really needed, it is usually badly needed; and, unfortunately, the work on which the future maintenance will be inadequate is just the work on which the original inspection and superintendence is inadequate.

It is the speaker's opinion that, in a great many cases, particularly for interior steel beams encased in concrete, no field paint whatever is necessary, and that the expense of painting such members would more properly be expended in the better painting of the members which really need it. Further—although it may seem contradictory

to the previous remarks—better results might often be obtained by using a \$5 man and \$1 paint, than a \$1 man and \$5 paint. Mr. Coombs.

As a result of some years of experience with a class of construction in which there are very thin members subject to an extreme and variable exposure, the importance of a revision in the common attitude toward the paint question appears to be of considerable importance. Such work may be regarded as an accelerated test, and as developing in a few years what may be expected in a longer period in heavier material, under incompetent supervision.

The speaker would like to ask Mr. Rights what he thinks the average price of shop paint is throughout the country, that is, considering the average of the total number of shops, not the average of the total tonnage.

Lewis D. Rights, M. Am. Soc. C. E.—The speaker, as a member of the American Society for Testing Materials, has attended the conventions at Atlantic City, has inspected the paint fence, and has listened to discussions regarding the values of inhibitors and stimulators. He has followed Mr. Sabin's paper and the discussions with a great deal of interest, with the hope that something would be brought out to help to clear up the situation, from the standpoint of one of the parties who is as much interested as any of the rest, if not more so. He refers to the bridge and structural manufacturer, who has the responsibility for the application of the paint, both in the shop and in the field, and is therefore a very important link between the paint manufacturer and the engineer and owner. Mr. Rights.

It may not be out of place to mention the conditions under which painting is done at the shops of the structural and bridge manufacturers. So far as the speaker knows, very few of the bridge shops have any adequate facilities for doing their painting inside a building, where the work can be done regardless of wet or freezing weather. Although it is true that during the most severe winter conditions painting is done occasionally in the finishing end of the shop, facilities there are so cramped that the quantity of painting done under cover may be said to be almost negligible. It is safe to say that practically all the structural steel painting is done in the open.

Some years ago, the owners of one of the largest shops carefully considered plans for covering their shipping yards and protecting them so that painting could be done at all times. The item of expense, however, was such that it was decided that the condition of the business would not warrant increasing the cost of painting to the owners.

The bureaus which inspect a large proportion of the fabricated steelwork have a rule that the finished material must not be painted until their men have had an opportunity to go over it. Frequently, it takes an inspector several days to cover the shops in his vicinity, and this means that the steel must sometimes lie in the open for a day

Mr. Rights. or so before it can be painted. Now, considering that painting cannot be done in either wet or freezing weather, nor can it be done until the work is inspected, it will easily be seen that the time allowed for doing the painting is very much reduced, and therefore, of necessity, the application of the paint at the shop must be a somewhat hurried job. In general, the class of labor doing the shop painting is fully as good as that employed in the other details of manufacture, and considerable care is exercised in endeavoring to see that all parts of the pieces are evenly covered.

In connection with the erection of structural steel, it has become the practice of many of the bridge manufacturers to sublet the painting to men who make a specialty of work of this class. It has been found that bridge erectors do not relish handling a paint brush, and better results have been secured with specially trained men. As the manufacturers are interested in producing a high grade of work, it has become the custom for them to buy the paint and ship it to the job in unbroken packages. It is then sold to the painting sub-contractor at an agreed price per gallon, and the work of applying the paint is inspected either by the owners' inspectors or by some one appointed by the bridge manufacturer.

The speaker would like to state here that the bridge manufacturer is not in favor of cheap paint. The fact of the matter is that he would rather put on a good paint, provided the cost of this good paint has been properly covered by the specification, and figured in his original bid. It will easily be seen that the more work the bridge manufacturer can put into a job, the greater will be his percentage of profit, and that, in addition, he will produce a more sightly structure. The bridge manufacturer carries at his shop several lines of standard paints, which are probably as good as can be bought under ordinary specifications, such as red lead, white lead, iron oxide, and some form of carbon or graphite. He is entirely willing to apply the best of these, as called for, or to provide a better paint, if required. What he most desires, however, is that the discussions which have been going on among engineers for the last few years will produce something in the way of standard kinds of paint suitable for each class of work. As matters now stand, it would seem that almost every engineer was working out his own theories of inhibition and stimulation, and that each one had a different variety of paint. Of necessity, the bridge manufacturer must order a somewhat larger quantity of paint than is actually required for each particular structure, with the result that the paint shop has the appearance of a medicine shelf, containing left-overs, each one strongly advocated by one individual engineer, and strongly condemned by all the rest of them. These left-overs will not remain fit for use very long, and are frequently a total loss to the manufacturer.

Years ago, a similar condition applied to specifications regarding the quality of structural steel. To-day, the manufacturer and the engineer have been able to get together, so that the same grade of structural steel is used for railway and highway bridges, buildings, and cars, and when the total tonnage is considered, the special-grade steel is a small factor. Cannot the paint manufacturers and the engineers get together on some such broad basis, and settle on paints which will be serviceable to the owner, without undue cost, which can be readily applied by the manufacturer, and which will fit 90% of the steel structures. "'Tis a consummation devoutly to be wished". Mr. Rights.

W. E. Belcher, M. Am. Soc. C. E. (by letter).—This subject is certain to hold the attention of a large proportion of the members of the Society, for all are looking for more information. The writer, however, confesses to a feeling of disappointment in reading the paper, particularly in regard to the very small number of paint materials selected by the author for consideration, and also because of his failure to cite from his own wide experience definite records of paint performances from which engineers might be assisted in forming their own conclusions. Mr. Belcher.

The writer would like to add the following random notes from his experience:

For several years a red lead paint was always specified by him for structural steelwork for buildings, following the example of many railroad and Government specifications. When inspecting the work done under practical shop conditions, the paint was found to be more or less unsatisfactory. There was a decided tendency for the formation of a hard sediment at the bottom of the paint bucket, and ordinary paint laborers never stirred the paint with sufficient frequency to obtain a uniform coat; this would upset fine calculation as to the number of pounds of lead per gallon of oil. There are on hand in most bridge shops several kinds of red paint which can hardly be distinguished from one another immediately after application, and might easily be substituted for one another—unintentionally perhaps—without subsequent detection. The writer has seen on detailed shop drawings the paint note, "One Coat Red," though the original specification had been quite elaborate. If specifications could be reduced to this simple form there would be no difficulty in getting what is called for. Another point which seems detrimental to the use of red lead is the fact that its color soon becomes dull after exposure, without regard to the number of coats applied.

The writer has always been prejudiced against graphite paints, on account of their extreme thinness and also because of the very conflicting arguments advanced by the manufacturers of the pure pulver-

Mr. Belcher. ized graphite paint, on one hand, and those of the mixed silica and graphite, on the other. A specification calling for Prince's mineral and for iron oxide to which graphite was added in small quantity for smoothness and color, was tried out with good results, and has since been followed. It overcomes very largely the objections above offered to red lead, and has generally good lasting qualities.

The preparation of a ground red lead paste mixed in oil would not appear to be entirely new, as one of the large paint manufacturers in Chicago submitted to the writer about three years ago a preparation called "Liquid Red Lead." This was a concentrated red lead paste to which, if memory serves correctly, a small quantity of silica was added, which assisted in an effective way against sedimentation.

This material was submitted in connection with a study of the subject of gas-holder paint. One manufacturer, submitting samples at that time, offered "Special Gas-Holder Red" which was recommended to fill every possible requirement, although the description of its manufacture was very vague. Investigation disclosed the fact that the manufacturers were as little certain of the desired ingredients of a gas-holder paint as the writer was, and, consequently, they put in a little of everything, including old house-paints, surplus paint stock, kettle scrapings, paint skins, etc., with enough red to color the mixture.

The writer is a firm believer in the theories of Messrs. Cushman and Gardner in regard to the corrosion of iron and steel; at the same time he must agree fully with Mr. Sabin that it is practically impossible for an engineer to use them in their present state as a guide for intelligent selection of a paint material. Has any manufacturer or manufacturer's representative ever been forced to admit that his particular combination of oils and pigments has been in any way discredited by these theories and experiments? The author's remarks regarding lampblack illustrate this point. We have learned that lampblack is in itself injurious. The manufacturer then states that he has ground it in a certain way which changes its action entirely.

The science of engineering is largely empirical, based on observations and experiments. The present discussion in regard to painting structural steel emphasizes the incompleteness of our record of experiments in this particular branch of the science. In the present situation, our rules are still in the nature of theories and deductions from limited personal experience.

Mr. Sabin. A. H. Sabin, Assoc. M. Am. Soc. C. E. (by letter).—It is not the desire of the writer to engage in unnecessary discussion, but it is proper to correct misstatements of facts. Mr. Gardner questions the statements made as to the Havre de Grace Bridge tests. It is true that in the paper as first published the percentage of silicates, etc., in Paint No. 6 was too low, but a correction was made at the time

of reading; the quantity of red lead was variously estimated at from 65 to 71% by different analysts. Paint No. 10, Mr. Gardner says, contained 10% of litharge, which was one analysis, but the same laboratory reported three other analyses of this paint showing 7.1, 6.8, and 8.5 per cent. Paint No. 11, he says, contained 5% of litharge; the four analyses give 3.51, 3.61, 2.99, and 4.14 per cent. The method used in the Contracts Laboratory* at that time gave results too low in red lead; the statement made by the writer was based on private information, and is probably accurate. Mr. Sabin.

Mr. Gardner says:

"The only authoritative paint tests, to determine the efficiency of single pigments as preservers of iron and steel, which have ever been made, are those by the American Society for Testing Materials, at Atlantic City, N. J., in 1908."

If this is true, the writer should have mentioned it in the paper; it seems, therefore, incumbent on him to explain his views as to the matter. In the first place, he objects to the statement that these tests were made by the American Society for Testing Materials. The plates were selected and cleaned, and the paints were designed, made, and applied by persons employed for the purpose by an association of manufacturers of mixed paints. The entire scheme and plan was theirs; and the only way in which that Society was mixed up in the matter was by accepting an invitation from the owners of the tests to inspect, by a committee, the prepared plates, as to the history of which they knew nothing except what the owners saw fit to tell them. According to Mr. Gardner, these tests show that no important single-pigment paint, and no simple mixture, such as can be made by an ordinary painter, gave results at all to be compared with the complex paints made by the people who designed the tests. Exception to this general statement may be taken in the case of a few pigments which are too costly to be used, especially the basic chromate of lead, which is described by Mr. Gardner as having "high litharge content", when in fact it contains no litharge at all.

The writer does not regard tests of this sort as authoritative or conclusive. He has before this been on a committee to inspect tests devised by Mr. Gardner, and has been impressed with the unfailing certainty with which they proved Mr. Gardner's contentions; and, for that reason, he can attach no value to them, however unprejudiced and competent may be the inspecting committee. This opinion is strengthened by the observations of A. W. Carpenter, M. Am. Soc. C. E., of the New York Central Railroad, and Mr. S. S. Voorhees of the Bureau

* The analyses reported from the Contracts Laboratory may be found in *Proceedings*, Am. Soc. for Testing Materials, Vol. VIII, Plates I to III.

Mr. Sabin.

of Standards.* Mr. Carpenter observed that in many cases three coats of paint on these panels were going to pieces in 20 months, though similar paints, according to his experience, would be better after an exposure two or three times as long, in New York City. Mr. Voorhees, who had been Chairman of the Committee on Protective Coatings from its beginning, said the Havre de Grace plates, after 5 years' exposure, were in better condition than those of the paint manufacturers' after 1½ years.

Mr. Toch's discussion contains two or three curious misstatements. He says that "every one of these red leads [on the Havre de Grace Bridge] was coated over with a carbon paint". According to the published analyses, as already referred to,† the pigments of Paints Nos. 10 and 11 showed only a trace of carbonaceous matter. He also says that his own paint was No. 12 and was the only one used in only two coats, all the others having three. The tables just referred to show complete analyses of each of three coats of paint on this section, the only one having but two being Section No. 1. It is reported‡ that:

"Three paints (Nos. 6, 10 and 11) in Class I, under each of the separate spreading rates, may each well be designated as excellent. * * * What differentiates these paints from all others under observation is the fact that while all the other paints except one furnish their best protection, such as it is, under the 600 sq-ft. rate of spreading and are generally markedly less effective under the thinner film rate, these three show such slight variation under different rates of application as to appear equally protective under either."

This disposes of Mr. Toch's statement that the report "describes as excellent nine paints, three of which are red lead". Mr. Toch also says: "every paint chemist knows that there are some red leads that contain nearly 1% of caustic soda"; as a matter of fact, all red lead is made in a reverberatory furnace, and it is plainly impossible for caustic soda to be present in such a product; if any were there, it would be quickly changed to carbonate; no one has ever seen any.

Mr. A. S. Cushman deplores the fact that the paper "entirely ignores the electrolytic theory of the mechanism of the reactions which lead to metallic corrosion." It may be that this criticism is justified; but the writer's view is that a discussion of the theory more properly belongs in a meeting of chemists rather than engineers. The writer believes that he has read everything Mr. Cushman has written on the subject which he plainly regards as peculiarly his own. The writer's ignorance, therefore, is not due to lack of instruction, doctrine, or reproof, but probably consists in a pernicious tendency to try to check

* *Proceedings*, Am. Soc. for Testing Materials, Vol. X, p. 90.

† Also Plate IV, Vol. VIII, *Proceedings*, Am. Soc. for Testing Materials.

‡ *Loc. cit.*, Vol. XIII, p. 338.

up theory with practice, which, however useful in practical work, is sometimes greatly disapproved by the theorist. What Mr. Cushman regards as the true theory of the corrosion of iron is, in the writer's opinion, that the action of water alone is all that is necessary for such corrosion to go on, and that the accidental presence of anything else is needless and not to be considered. This theory involves the supposition that in all water there is a portion which is ionized; that is, part of the hydrogen exists uncombined with the rest of the molecule, an unstable condition, which results in the formation of new compounds when another substance, as iron, is present. This theory, as regards the corrosion of iron, was first proposed by Mr. W. R. Whitney, now Director of the Chemical Laboratory of the General Electric Company, several years before Mr. Cushman adopted it.* Mr. J. N. Friend, who is Principal of a technical college in England, has been the chief opponent of this theory; he has been supported by grants from the Carnegie Institute, of Washington, and has contributed papers to the *Proceedings* of the Iron and Steel Institute on the subject, in which he believes that he has experimentally disproved the theory in question. A full discussion may be found in Mr. Friend's book.† Mr. Whitney, in his original paper, pointed out the enormously greater theoretical efficiency of carbonic acid, which is probably not denied by any one; and as this acid is a normal ingredient in air and rain, and all natural waters, it has always appeared to the writer that the pure-water theory of corrosion was of doubtful practical value. Further, in view of the fact that no one doubts the existence of electrical disturbances coincident with chemical action in general, and especially as it has long been known that electric instability is provocative of corrosion of iron, it seems to be an arrogant assumption to claim the exclusive application of the term electrolytic to the very limited theory propounded by Mr. Whitney, a restriction never proposed by him. Mr. Sabin.

The fact that iron does not rust in certain solutions was known many years ago. Such facts are explained, some of them, and perhaps all may be, by the modern theories of chemical action; and Mr. Cushman has laudably attempted to apply such knowledge to the paint industry. The question in regard to this is: are his inventions, whether patented or not, of any practical value? The only answer the writer can give is that they do not appear to be. Mr. Cushman seems to believe that a linseed oil film absorbs water from the air, that it is in a sense hygroscopic, and that part of the solid particles of pigment in a dried paint film are really in solution; that this water, which is to all intents in solution in the solidified oil, acts on the

* *Journal*, Am. Chemical Soc., 1903, pp. 394-406, Whitney; and *Proceedings*, Am. Soc. for Testing Materials, 1907, pp. 211-228, Cushman.

† "Corrosion of Iron and Steel," 1911, pp. 44-67.

Mr. Sabin. iron and causes corrosion, and if some slightly soluble chromate, for example, is in the pigment, it will be in solution in this water and will prevent or forbid, or, as he says, "inhibit", corrosion. It is a very pretty theory, but does it work?

In reply to this, attention is again directed to the official report.* After 5 years' exposure, nineteen test panels, representing as many paints, were removed from the Havre de Grace Bridge, photographed, and then the paint was thoroughly cleaned off and the surface of the metal inspected for rusting. Excepting a few spots, where the coating was obviously injured accidentally, the surface of most of these plates was in exactly as good condition as when first painted. According to Mr. Cushman's theory, all these plates should have rusted under the paint; none of the paints used contained what he regards as efficient inhibitors, and some of them contained large quantities of carbon, apparently lampblack, which Mr. Cushman says is the worst stimulator there is. The paint in all cases was thin, being three coats at a spreading rate of 900 sq. ft. per gal., or a total thickness of 0.005 in., and had been exposed to the moist salt air from Delaware Bay, at the mouth of the Susquehanna River, for 5 years. The writer thinks this proves that a good paint, properly used, keeps out air and water; and, as long as it remains intact, it protects the metal; when it has deteriorated so much that it has holes through it, the air and water get to the iron, which begins to rust. As long as there are no holes, no inhibitor is needed; when there are holes, no inhibitor will do any good. No doubt there is a difference in the value of pigments; perhaps a soluble sulphate acts to some extent by conducting water through a film, and perhaps acts on the oil itself; perhaps a substance like carbonate of lime forms a soap with the oil and injures it in that way; but of more practical importance is the greater or less surface attraction between the pigments and the oil; in this respect the differences are great and characteristic. To illustrate this, let us remember that when one end of a small glass tube is put in water the water rises, as we say, by capillary action, in the tube; but, if mercury is substituted for water, the surface about the glass is depressed as though it were repelled. These are cases of surface tension and attraction. In an exactly similar way, some pigments have more or less surface attraction for oil. The most remarkable, and at the same time the most important, case is where, in the manufacture of white lead, the pigment is subjected to prolonged washing with pure water, then allowed to settle, and the wet pulp is agitated in a mixer with raw linseed oil. Although the oil is lighter than water, it displaces the water, mixes with the pigment, and forces out the water, which runs off the top, leaving the lead and oil almost free from moisture. Red lead may be treated in a somewhat similar man-

* *Proceedings*, Am. Soc. for Testing Materials, Vol. XI, pp. 175-180.

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ner, but of course it is not, as it is a furnace product. As far as the writer knows, no other pigment will act in this way; in most cases the presence of a little water in the pigment greatly obstructs its proper mixing with oil; but pigments differ greatly in this attraction for oil; which may partly account for the widely varying quantities of oil required by different pigments. To give an entirely different illustration: it is common knowledge among all paint makers and users that 1 quart of turpentine will thin a batch of paint as much as 2 quarts of linseed oil, because of the greater fluidity, or less viscosity, of the former. However, if we make a stiff paste with any ordinary pigment, as white lead, we require a certain quantity, say 8% of oil; if we substitute turpentine for half of this oil, it will take, not 2%, as might be thought, but 6 per cent. This remarkable fact leads to the consideration that the paste is a plastic body rather than a viscous one, and the function of the liquid is to stick the solid particles together, which the oil does much better than the turpentine, the surface attraction of which for the lead is very low; but, when more oil has been added, to make paint, which is a viscous but not a plastic body, turpentine is more efficient as a thinner than oil. In a similar way we account in part for the advantage of proper cleaning of steel before painting. It is well known that any ordinary oil easily wets clean iron and steel, spreads over its surface, and is removed with difficulty; but, if the surface is fouled with anything which does not attract the oil, the latter when brushed out into a very thin film—and paint films are very thin—may break and leave holes, or pores, through which the atmospheric agencies get to the metal.

If, now, the pigment is one which has a great surface attraction for the oil, it is obvious that, when the paint is brushed out to a thin film, the presence of these solid particles in vast numbers will tend to hold the film together, to make it in fact tougher, so that it will be less likely to break into holes; and this is one reason why fineness is one of the most desirable qualities in a pigment; and why such a paint brushes out easily into a thin film, or, as we say, has good working quality. It is also a reason why a paint of good working quality (if it has no counterbalancing defects) is a good protective paint, for it makes a continuous film. Conversely, if the attraction between the pigment and the oil is slight, the paint is less likely to be satisfactory in any respect. Nor is this advantage likely to disappear when the film hardens; for the particles which attract the oil are likely to be more tightly cemented into the mass and form an element of strength rather than weakness; and if, in addition, the pigment is one which shows very little, or perhaps negative, attraction for water, it is likely to show great resistance to atmospheric action.

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Linseed oil is an excellent insulator against the transmission of electric currents; and some pigments are themselves non-conductors, though others are the contrary. As it is generally agreed that electric tension is an effective cause of corrosion, it is desirable that paint for metal should contain pigments which will tend to insulate, rather than to break down the insulation given by the oil itself. This is a very different thing from Mr. Cushman's theory that certain pigments embedded in a cement of dried oil will act, in conjunction with pure water (which, in the writer's opinion, could not possibly get there), as a primary battery bringing the iron into solution. Mr. Cushman has elsewhere described the action of certain pigments as like a poison to iron; if correct, which in some cases at least may be doubted, it is because of their unfavorable relation to the oil, which renders them incapable of making good films, and the remedy should be looked for in this direction, rather than by his favorite prescription of a homeopathic dose of some inhibitor.

It is well known that some paints, including some varnish paints, and especially red lead in oil, adhere to iron and steel much better than others. Probably the mixture of a pigment in the vehicle affects its surface attraction to the metal. When we consider that often as much as one-third of the volume of a paint is composed of these solid particles, and that, in the case of a really fine pigment, there are from 25 to 50 particles overlying one another in a film 0.002 in. in thickness (this supposes them to be 0.00001 in. in diameter, which is within the truth), it may be believed that the action of such a mixture is very different from that of oil alone.

Mr. Carpenter's discussion is interesting and valuable. In Table 1, showing his red lead experiments, it is notable that the quantity of paint used in the second series was 6.3% more than in the first (owing to the use of more pigment), but the surface covered was almost 30% more; and in three cases out of five the working quality of the paint was better, and equal in the other two. If a mixture of 30 lb. of red lead to 1 gal. of oil will work better and cover 28% more surface than a 24-lb. mixture, the fact is worth knowing, and should lead to considerable corrections in common practice.

The relative fineness in these tests was determined by the weight in grammes per cubic inch. The writer does not regard this as a safe method. Certainly, a very fine red lead, after being packed in a barrel in the usual way and then sifted to make it perfectly loose and open, will weigh several more grammes to the cubic inch than it did before being barreled; and it is probable that the volume depends as much on the friction between the particles as on anything, and this, again, may depend on their electrical condition, shape of particles (red lead particles differ in shape according to the material from which they are made, some of which is crystalline), and probably many

other unknown causes. At all events, in the laboratory with which the writer is connected, the best-known volume apparatus gives results so widely and irregularly at variance with those of a more nearly absolute method which is checked by actual microscopic measurement, that it is not regarded by the writer as entitled to much consideration. It is possible that a fine red lead may contain considerable litharge, if it is taken out of the oxidizing furnace too soon; also, it is, at least theoretically, possible to oxidize highly a somewhat coarse material; but, in general, the finer the material the higher is the oxidation. **Mr. Sabin.**

Mr. Carpenter's criticism of lack of definiteness as to the composition of the Havre de Grace tests has already been met by the figures given; attention is also asked to Mr. Wagner's discussion giving his experience with red lead ranging from 96.48 to 98.63% for 5 years, with satisfactory results. It should not be forgotten that, in the Havre de Grace tests, at least two of the red leads were what may fairly be called high grade, and were furnished by some of the largest and most experienced paint manufacturers in the country; and it is fair to infer that these makers desired to make the best possible record, and therefore that they had evidence, satisfactory to them, that this was the best material to use; as it proved to be.

As to why it should be better, the writer must admit that the reasons given in the paper are partly theoretical; but he does not see why he should be denied adopting a theory any more than other people, particularly as this one has nothing novel or original about it, is generally held by the trade, and has been forced on him by the constant experience of a quarter of a century. Age, however, does not make a theory right, any more than it does a man; but it entitles it to respectful consideration; it has successfully served a useful purpose for a long time. There is a broader ground, however, on which all can meet; for all agree that more than half the battle is in getting the paint applied to a clean surface in a smooth and uniform coating; smooth, because resistance to wear is much increased by the absence of ridges, lumps, and grains in the surface, and uniform, because thick masses of paint are waste, and thin places are weak. As Mr. Coombs puts it, a $5 man with a $1 paint is better than a $1 man with a $5 paint. Now the whole history of the improvement of red lead as a paint has been of this nature: to please the consumer, it has been made continually finer in texture, so as to be free from roughness and disposition to run, and lower in litharge, so as to be less active to oil and therefore more manageable. Tendency to settle is not so much a matter of specific gravity, but is caused by coarseness; no paint settles less than white zinc and white lead, which are very heavy but very fine; and no paint is more refractory in its general behavior than a low-grade, red-lead paint which has begun to get thick and viscid. It accords, then, with general experience, that a red lead which consists entirely of impalpably

Mr. Sabin. fine particles and has about the same relation to oil that white lead has, will be easily applied and form a smooth and uniform coat; and that, irrespective of the theoretical questions involved, it will practically be better on the average and last longer. This, it is submitted, is in accord with all experience with painting in general.

One more observation is suggested by Mr. Carpenter's discussion which is that it is difficult to determine spreading capacity in a panel test. The painter instinctively tries to spread all alike, so as to give each paint a fair chance; the more conscientious he is, the more his results may vary from what are actually secured in working under average conditions. It may be doubted whether a test of less than a barrel of paint is of much value in this regard.

In conclusion, the writer would explicitly deny the charge, made by Mr. Cushman and his associates of the paint manufacturers' organization, that he claims to have said the last word in respect to the protection of steel, and that there is no need of further investigation of paint problems. Such a charge will not be taken seriously by any member of the Society. Another assumption, that the paper was designed to create an interest in red lead, is really based on the fact that every one is interested in that substance, and when any one has anything essentially new to say about it, whatever else he says is likely to be neglected. This very fact, that it is a widely known and valuable material, made it proper to mention, in a paper of this character, any important improvement in its manufacture or use, and the fact that the writer is able to speak authoritatively of its composition and character seemed a good reason for doing so. It is not a perfect paint; there is no paint suited for all uses; but that is no reason why we should not know as much as we can about any of them, particularly those which we use the most.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1301

TOPOGRAPHICAL SURVEYS MADE BY THE AMERICAN SECTION OF THE INTERNATIONAL BOUNDARY COMMISSION UNITED STATES AND MEXICO.*

By W. W. Follett, M. Am. Soc. C. E.

With Discussion by Messrs. William B. Landreth, W. N. Brown, N. T. Blackburn, Leonard S. Smith, and W. W. Follett.

Introduction.

During the winter of 1910-11, the International Boundary Commission between the United States and Mexico, of which the writer is Consulting Engineer for the United States and Mr. E. Zayas for Mexico, made a survey of the Rio Grande from Roma to the Gulf of Mexico. This survey covered an area about 140 miles long and from 2 to 3 miles wide, including within its limits the territory which is subject to the erosive action of the river. The purpose of the survey was to obtain data from which a map could be made on a scale of 1 in 5 000, which would be accurate as to general dimensions and the location of fixed points, and would show, within reasonable limits, the location of the topographical features.

The country traversed was practically a level plain with an escarpment, from 1 to 3 m. high, on each side of the river, which marked

* Presented at the meeting of December 3d, 1913.

the limit of ordinary overflow and also, approximately, the margin of the erosive area of the river. These escarpments were from less than 1 mile to 2 miles, and, in extreme cases, 3 miles apart. Practically the whole area between them has been at some time, or may be in the future, the bed of the river. There were no hills suitable for triangulation. The main wagon road on the north side of the river was, for the greater part of the way, on the bench above the limit of overflow line. There was a thick cover of brush and trees on three-fourths of the area to be surveyed.

It was the purpose of the survey to locate all old channels and breaks in the surface between the two escarpments, including, of course, the river, with its high and low banks, all wagon roads, towns, ranches, irrigation or drainage ditches, levees, railroads, etc., both below and above the escarpments, within the limits of the map, together with a sufficient number of elevations to locate meter contours.

The river has cut off many bends, forming what are locally known as "bancos." Around these, the Commission, at various times, has laid traverses which mark, approximately, the center of the old channel abandoned by the river when it cut off the banco. These traverses form the property lines between the owner of the banco and the adjoining mainland proprietors. Their corners are marked by substantial timber posts, and there has been built on each banco a concrete monument to which the traverse is tied. All this work has been done accurately, but, when the field operations began, in the fall of 1910, the different banco surveys were not tied together.

Methods.

The methods used in making this survey were as follows:

First. A precise line was located and measured, which, for the greater part of the way, was along the main wagon road. At intervals of from 2 to 3 miles, small concrete reference points were built and accurately located.

Second. From this precise line spurs were run to the monuments on the bancos.

Third. Stadia traverses were run, starting from, and closing on, some precisely located point, or from and to stadia hubs which had already been tied to precise points.

Fourth. A primary line of levels was run from Roma to the Gulf, and the elevation of each reference point was accurately determined.

Fifth. Ordinary levels were run to determine the elevation of each hub on the stadia lines.

The precise line, the banco ties and traverses, and the primary levels were run jointly by the two sections (American and Mexican) of the Commission; that is, the work was first done by one section and then checked by the other, the notes were compared, and an agreement was reached between the two consulting engineers. The stadia lines were run independently and the maps compared.

In order that the reader may understand the operations, Plate XVII, a small section of the original 1 in 5 000 map of the American section, is submitted. The original map was printed in colors, so that it is much more easily read than this sample. It also showed meter contours and many elevations which are here omitted, as are also the latitude and longitude and latitude and departure lines, both of which are shown on the original.

The traverses around the bancos (Tahuachal, La Isla, and Tahuachalito) and the lines having angle points marked A, B, or C, were chained. The "A" is the precise line. The T, R, and M points are stadia stations. A study of the lines will show how they were tied together and to the accurately located points—either banco corners or A, B, or C hubs.

The heavy broken lines show the channel of 1897-98. It is taken from a survey and map made by the writer for the Boundary Commission. Comparison of it with the river as found in 1910-11 gives an idea of how much the river shifts by erosion and deposit. East of La Isla, there is seen a banco in the making. In a few years more, possibly only 3 or 4, the river will go through the neck near R-1 569, and a new Mexican banco will be cut to the American side of the river.

The hatched line marked "limit of ordinary overflow," is the escarpment before mentioned. It is an important line, for besides showing where ordinary floods stop, it marks quite closely the erosive limit of the river, and shows a change in soil and vegetation. Below it is recent alluvial deposit with wet-land growth. Above it is considerable clay and a semi-arid growth, such as cactus, mesquite, and other thorny trees and shrubs.

When fully organized, the American section had five transit and two level parties in the field. One transit party laid out and measured the precise line, one ran the banco ties and located some banco traverses which had not been run, one took by stadia the river with its adjoining topography, one obtained Texas topography, and the last one did the work on the Mexican side.

One level party ran the primary level line, carried levels over the river topography hubs, and took rough cross-sections of the river bed opposite each hub. The other one ran levels over the Texas and Mexico topography hubs and hunted high-water marks.

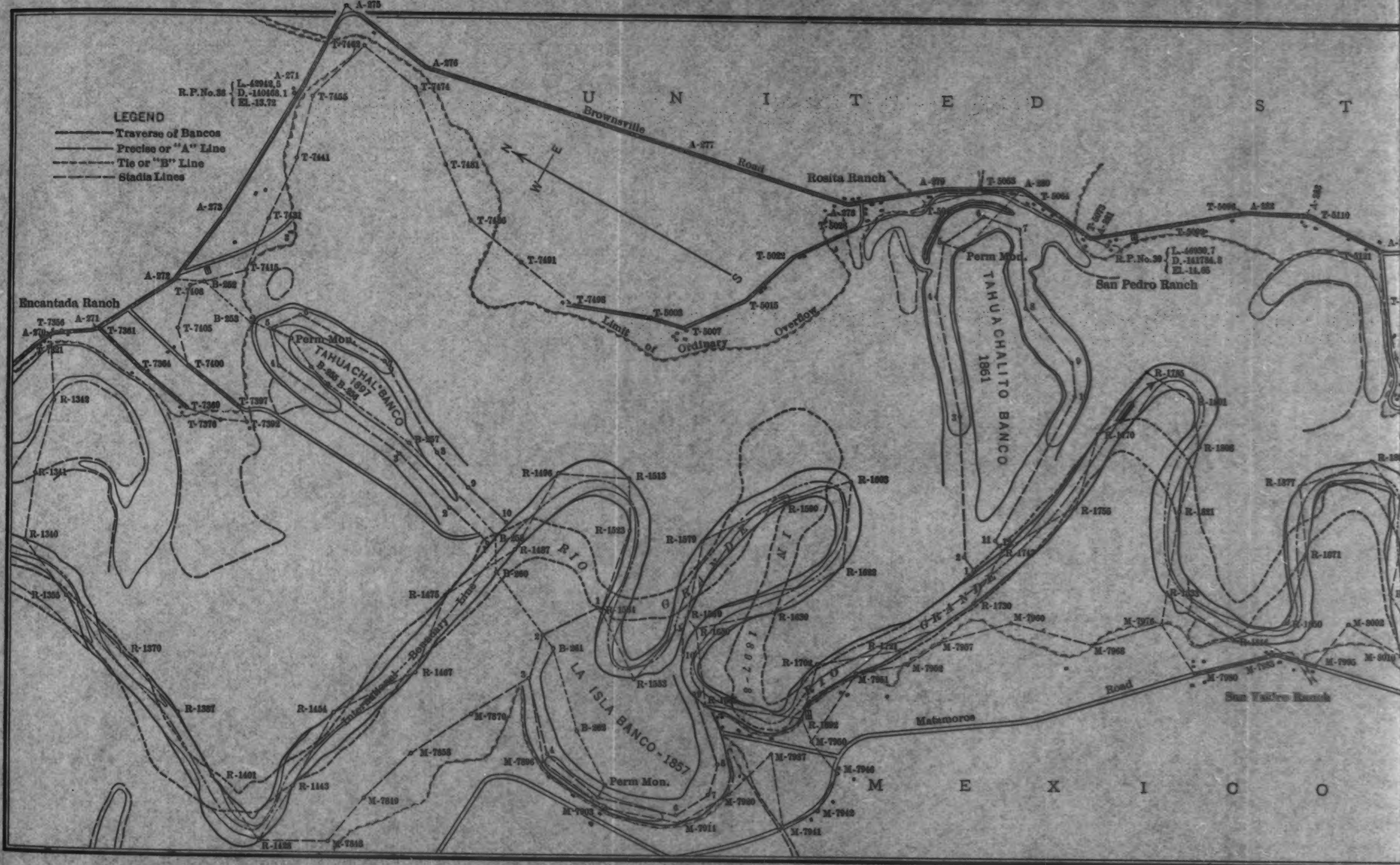
In the office, a computer kept up the computations of the precise line, banco ties, and new banco traverses, referring all by latitude and departure to an assumed initial point in Roma. A draftsman, working at night, platted, by latitude and departure, the precise points, banco ties, and traverses, and, by protractor, the stadia lines which had been run during the day. Another draftsman, working during the day, platted the topography as fast as the books came in. In this way, close check was kept on the work which was being done. Failures to close on stadia traverses were detected at once, and the omission of any topographical features was soon noted.

The survey and map were controlled by the precise line. In computing this, the observed azimuths were corrected by adding 1.6″ for each 100 m. easting, measured from center to center of the courses. This is the amount of the divergence of a parallel of latitude from a straight east and west line in Latitude 26°. By thus correcting the azimuths, the parallels of latitude are represented on the map by straight lines, and the meridians are drawn at right angles to them. Of course, this did not make a correct geodetic map, but represented the surface as being a plane.

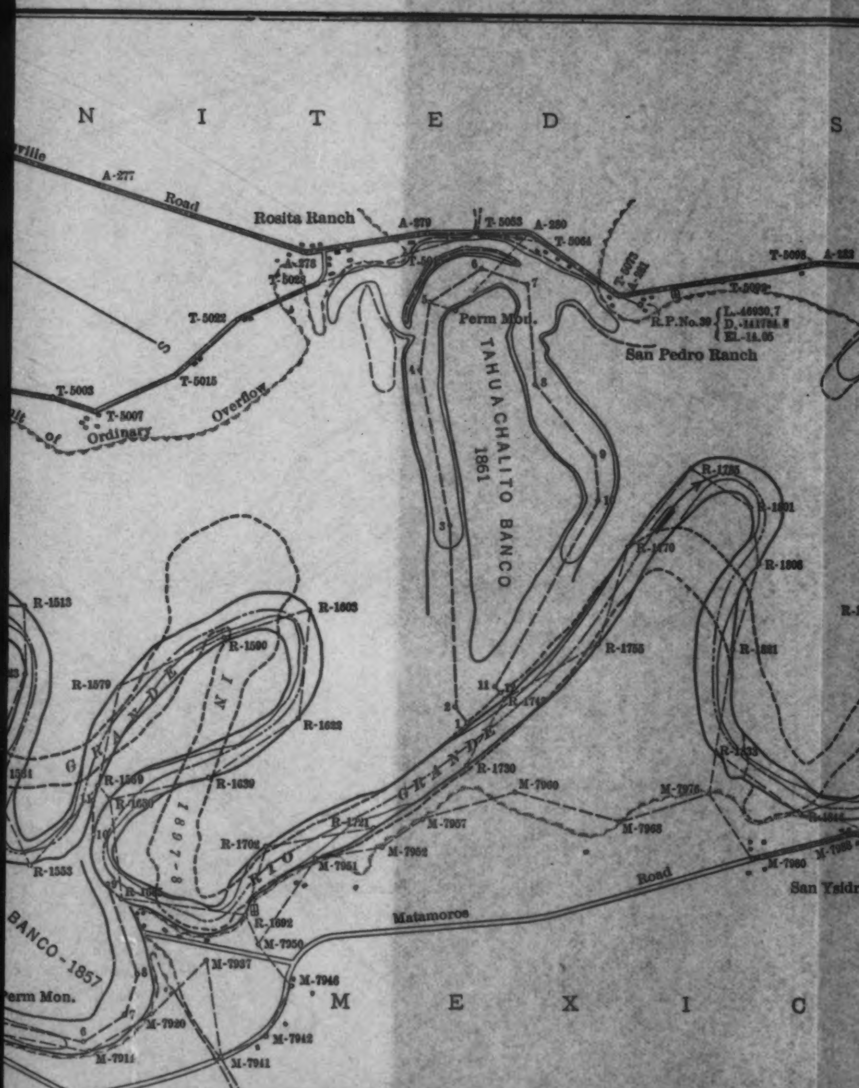
With so many parties in the field, setting stakes, it was necessary to have such a system of marking instrument points and keeping notes that no confusion would result. The following scheme was adopted:

The precise line was called the "A" line, and the stakes were marked consecutively from 1. This line was double-chained, the angles repeated, and the latitudes and departures computed in camp, after the azimuths were corrected for easting.

The ties to banco monuments were marked "B," with consecu-

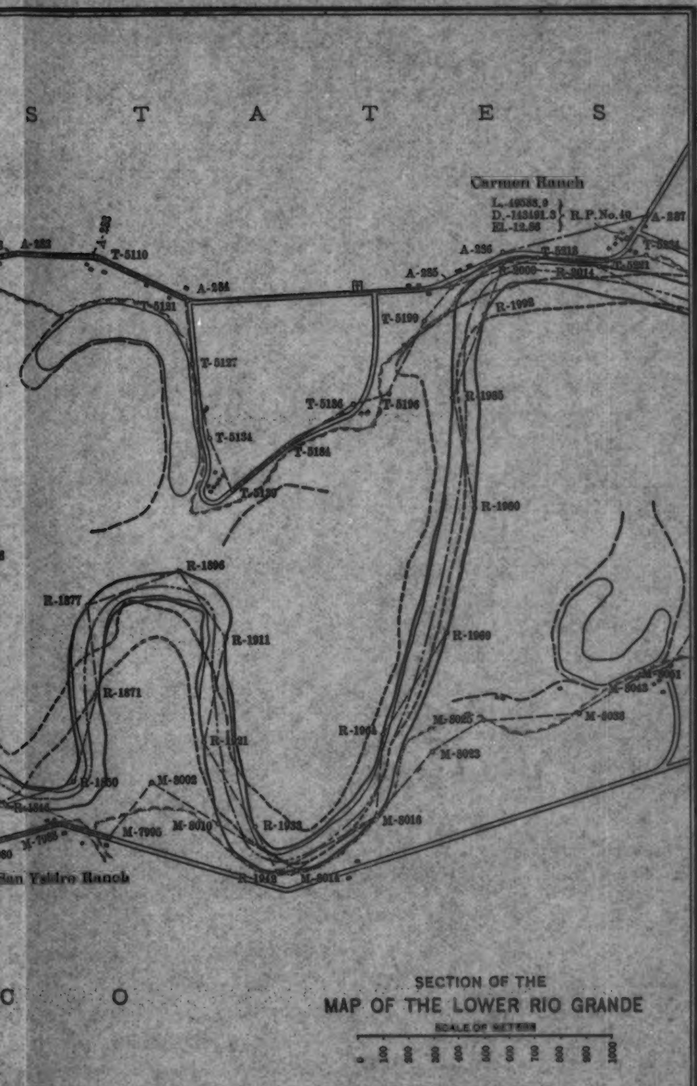


LEGEND
Traverse of Bancos
Precise or "A" Line
Tie or "B" Line
Stadia Lines
U N I T E D S T
M E X I C O
Brownsville Road
Rosita Ranch
Encantada Ranch
San Pedro Ranch
TAHUACHAL BANCO 1897
TAHUACHALITO BANCO 1861
LA ISLA BANCO - 1857
RIO GRANDE
International Boundary Line
Limit of Ordinary Overflow
Matamoros Road
Perm Mon.



N I T E D S
Road
A-277
Rosita Ranch
A-279
T-5053
A-280
T-5064
A-278
T-5023
T-5022
T-5015
T-5003
T-5007
Ordinary
Overflow
Perm Mon.
TAHUACHALITO BANCO
1861
T-5098
A-283
T-5099
R.P.No.39
L.-46930.7
El.-14.05
San Pedro Ranch
R-1513
R-1603
R-1590
R-1579
R-1622
R-1639
R-1569
R-1630
R-1553
R-1702
R-1721
R-1730
R-1755
R-1770
R-1785
R-1801
R-1806
R-1831
R-1833
GRANDE
RIO
1897-8
M-7960
M-7976
M-7968
M-7957
M-7952
M-7951
M-7950
M-7937
M-7946
M-7942
M-7941
M-7930
M-7914
M-7980
R-1692
Matamoros
Road
San Ysidr
BANCO-1857
Perm Mon.
M E X I C

PLATE XVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVII, No. 1301.
FOLLETT ON
TOPOGRAPHICAL SURVEYS.



tive numbers from 1. These lines were double-chained, the angles repeated, and latitudes and departures computed in camp.

Auxiliary closed traverse lines around any of the new bancos were marked "C," with consecutive numbers from 1. As these were closed lines, they were chained only once and the angles were usually measured only once, although they were sometimes repeated. The latitudes and departures were computed in camp.

As the traverses to all bancos had been carefully chained, tied to the permanent monument, and computed, when a man in the field found a banco corner or a hub with a witness stake marked A, B, or C, he knew that he had a point having a known latitude and departure, and that he could tie to it. He also knew that the azimuth of the line, whatever it was, was known within a minute, so that he could check his azimuth.

On the topographical work, every shot was given a number, whether it was a side shot or to a hub. The river stakes were marked "R", the Texas stakes "T", and the Mexican ones "M". To the river were assigned the numbers 0 to 5 000, to the Texas stakes, from 5 001 to 7 500, and to the Mexican ones, from 7 501 to 10 000. When a man used up the numbers assigned to him, he started over again. Surveys for taking the topography of new bancos used numbers from 10 001 up. It followed, therefore, that the number of the stake, even without the letter, showed what line it was on.

In recording instrument points, the original number marked on the stake was always used, no matter how many other lines ran to that point. This prevented confusion, and when a non-consecutive number appeared in the notes, it always drew attention to the fact that a tie of some sort to some previously located point had been made. The rule was followed rigidly, especially when tying to points set by the Mexican Section.

Results.

The first questions which an engineer asks when looking over any work of an engineering nature, are: What methods were used, what results were obtained, and what was the unit cost of the work? The first two questions can be fully answered. As no separate accounts were kept of the different kinds of work, only an approximate answer can be given to the last.

In all the work, joint and otherwise, it was constantly borne in mind that a reasonable degree of accuracy was a desideratum, but that no refinements should be introduced which would add greatly to the cost without increasing the practical value of the work. In the following discussion, the deviation in this respect from the methods sometimes used are fully set forth.

For convenience of ready reference, the limits of error to which it was attempted to work, are given:

Precise Line.—Angles should not be more than 2″ of arc in error. Distances should not differ more than 1 in 20 000, or 5 mm. to the 100 m.

Tie Lines to Bancos.—Angles should check within 20″ of arc. (They really checked much closer than this.) Distances should not differ more than 1 in 10 000.

Primary Levels.—Elevations should check within

$$0.01 \text{ m.} \sqrt{\text{distance, in miles, between benches.}}$$

Stadia Work.—No definite limit was specified, but, whenever a platted closure was so large as to indicate an error in reading distances, the line was re-run. Roughly, the allowable error in platting was 2 m. to the 1 000. Azimuth should check within 5′ of arc.

Precise Line.

The purpose of the precise line was to furnish a base to which all stadia lines could be tied, and to determine the relative position of the reference points and permanent monuments. It was the intention to make the precise work so accurate that it would take the place of what would be called a secondary triangulation. There was no attempt at extreme accuracy, but care was taken to obtain uniform work, and the best result to be had with an engineer's ordinary No. 1 transit.

The country was peculiarly fitted for chaining, being usually as level and smooth as a floor. This is seen from the fact that, out of 411 courses, on only 26 was it necessary to make corrections for portions of the line for differences of elevation.

The party on the precise line consisted of a transitman and two chainmen, and several laborers who acted as flagmen and axemen. The instrument used was an ordinary Buff and Berger No. 1 transit, made in 1885, with the plate graduated to 20′ and the verniers

reading to 20″ of arc. This instrument had been in ordinary use for 25 years, and had been returned to the makers once for overhauling. A Roe, 100-m. tape was used. It was graduated to meters, with end meters in tenths. A small piece of old steel tape was also carried, in order to read fractions of a meter. A cheap thermometer in a wooden case was used for taking temperatures. The chain lengths were marked by steel pins $\frac{3}{16}$-in. in diameter. The tape was supported throughout, that is, laid smoothly on the ground. The sight poles were ordinary 8-ft. transit rods. Another 100-m., Roe tape was kept in camp and used as a standard. With it the tapes in use by both sections of the Commission were compared frequently.

The programme of work was about as follows: The two chainmen, under the direction of the transitman, with three or four axemen, would go ahead and lay out the line. They usually followed the main road. This was sometimes quite crooked, with dense thorny undergrowth on each side, and it required considerable experimenting to get the transit points in places such that the brush-cutting would be reduced to a minimum and with the proper distance of 100 m. or more between points. No effort was made to obtain very long sights, as it was found that the boiling of the air was likely to be so great that the transit rods could not be seen. Only a few sights were taken which were more than 1 000 m. long; the average length was 584 m. When the chainmen had ranged in a line and cut away enough brush to obtain a sight, a hub, about 2½ in. in diameter and as long as could be forced into the hard ground without brooming, was driven flush with the surface and a hollow-headed tack was put in its center, a witness stake and three guy stakes were driven, and the witness stake was marked with the consecutive number. The line was given the letter A, and the stakes were marked consecutively. A-1 was in the main road near R. P. No. 1, 2 miles above Rio Grande City. From there the numbers ran toward Roma until A-26 was set. Here, a junction with the Mexican line was made on their Z-410 stake, and their numbers were used into Roma where Z-399 was found, from which R. P. "E" was set.

A-27 was set toward Rio Grande City from R. P. No. 1, and it was from A-385 that R. P. No. 55 was set on the shore of the Gulf.

Starting from R. P. "E", there were 12 courses down to A-26; from there to R. P. No. 1 there were 26 courses, and from R. P. No. 1 to

the Gulf there were 373 courses, or a total of 411 courses in all. The reason that the number of courses exceeds the points marked A is that 17 of the reference points and one hub set by the Mexicans and marked M-11 were used as angle points on the A line, and that four A numbers were used on spur lines to reference points. The usual rule was followed of giving a hub only one number, and that the one it first had.

After 3 or 4 km. of line had been located, the axemen were left to finish the clearing, which was done with great care so that the tape would lie smooth and straight, and the chainmen started to measure with the 100-m. tape and 11 steel pins (the surveyor's ordinary number). There were long leather thongs in each end of the chain for holding it. The head chainman carried the forward chaining book, spring balance, and thermometer. When a full chain was reached, the thermometer was laid on the ground face up, in the sun, if it were shining, so that the temperature would be the same as that to which the tape was exposed. The tape was pulled up to a strain of 15 lb. and held there until the rear chainman, who was centering the end mark over the tack, called "all right". The pin was then stuck, the tape eased off and then pulled up again, and the position of the pin checked. The head chainman then entered the chain length and the temperature in his book. When the next hub was reached, a pin was set at the last full meter, and the meters were read. Then the two chainmen changed places, and the plus was read again. The fractional meter was read by a piece of Chesterman steel tape, the number of entries in the book were verified by counting the pins, these were again bunched, and a new course was started.

The notes read as follows:

219	100	82°	311.624
to	100	83°	+ 0.04259
220	100	83°	311.66659
	11.624	83°	

The work at the right of the foregoing was done by the transitman, and was the reduction to 62°, with a coefficient of 0.0000065 per degree. This reduction was here plus, and for 83° — 62° = 21 degrees. The tape was standard at 62 degrees.

A run ahead was made for half a day. Then the forward chaining book was given to the transitman, and the backward one obtained; then the measurements were made again, but in the opposite direction. The notes for the foregoing course then read:

220	100	88°	311.620
to	100	88°	+ 0.05273
219	100	88°	311.67273
	11.620	88°	

The correction for temperature in this case was for 26 degrees. The mean of the two measurements was 311.66966, and the difference between the two was 0.00614. As the allowable difference was 0.016 (5 mm. per 100 m.), the chaining checked, and the adopted distance was: A-219 to A-220 . . . 311.67.

It is not assumed that chaining can be done to five decimals, but they were used simply for convenience in computing the correction for temperature. The chaining was actually done to millimeters, but the adopted distances were only carried to centimeters. The writer does not approve of recording the results of any work in figures which indicate an accuracy beyond that which the actual working conditions will give.

This chaining approximates quite closely in accuracy to that done in measuring secondary bases, where it is customary to use some mechanical arrangement, such as a lever, to hold the end of the chain firmly and perfectly still. Better work should result from such an appliance, but its use consumes time. It appeared that results sufficiently accurate could be obtained without it, and so none was used.

Elongation of Polaris.—The elongation of Polaris was usually taken at each camp, these being from 6 to 8 miles apart. The azimuths of the stadia lines were checked on these elongations and also that of the precise line, if the latter were near enough. As the camp was always on the river, in houseboats handled by a gasoline launch, it sometimes happened that it was a mile or two from the precise line. In that case, an elongation was taken from an "A" hub, or the azimuth of the precise line was not checked until the next camp was reached.

When an elongation was to be taken, a point for the hub was chosen and a line about 100 m. long was cleared along the proper

azimuth. Some 15 or 20 min. before the computed time, the transit was carefully set and leveled, pointed to the polar star, and clamped. The star was followed until it was seen that it had nearly reached its elongation. Then a hub was driven, and a chaining pin (with a sheet of white paper back of it and a light back of that) was held on the hub. The cross-wires of the transit were illuminated by the light of a candle in a tin can directed on a piece of white paper with a hole in the center, which was fastened by a rubber band on the shade of the object glass. The transit was again turned to the star and kept there until the latter was seen to drop (or rise) along the wire. Then line for tack was given, on the pin; the transit was reversed and set again on the star and another point set on the hub, and the tack was put half way between the two. For the whole time of the survey the bearing of the elongation was assumed to be 1° 18′ (azimuth, 358° 42′ for western elongation and 1° 18′ for eastern).

In the writer's opinion, more accurate results can be obtained in this way than in observing Polaris at any time, referring it to some hub previously set by reading the plates, and then computing the azimuth of the line from the transit to the hub. The two principal objections to this latter method are, the uncertainty which may exist as to the exact time of the observations and, more important still, the use of a light around the transit for reading the verniers. Where there is light there is heat, and the heat expands one side of the transit, thus affecting the verniers. Both methods have been tried, with the invariable result that the observation of the elongation was the more accurate.

Measuring Angles.

The transitman worked ahead of or behind the chainmen, whichever was more convenient. He had two flagmen, each equipped with an 8-ft. transit rod having three guy wires on it, and a plumb-bob. The rod was set up on a hub, guyed to the stakes before mentioned, and plumbed. It was found that, with his hands, a man could not hold a rod so firmly that the angles read to it would check within the desired limit.

Starting at a hub from which the elongation of Polaris had been observed, the transitman set his left-hand or "A" vernier to read zero, and read the right or "B" for seconds, recording both readings. He then noted on which side of the traverse line was the angle which

was less than 180°, and set his instrument on the left-hand stake, clamped the plate, and turned to the right-hand stake; he then read and recorded vernier "A", simply as a check on the later work. Unclamping the lower plate, he turned to the left-hand point and accumulated the angle twice more on the plate; then he reversed the telescope and accumulated the angle three times more, read and recorded both verniers, observed whether his line of sight was still pointing to the right-hand rod, and, if not, set it back with the lower slow-motion screw, swept the outside angle three times, erected the telescope, and swept it three times more, ending with the instrument pointing to the left-hand hub. He then carefully read and recorded both verniers. If there had been no slip in the instrument and his pointings had been perfect, his verniers would read the same at the close as at the beginning.

The following shows the notes of the angle at A-219:

Left-Hand Page.

Instrument.	A.	B.	Mean.
Inst. at A-219	00′ 00″	00′ 00″	
Δ A-218-A-220	124° 16′ 40″		
	25° 39′ 30″	39′ 30″	
	00′ 20″	00′ 20″	
Needle N. 86° 05′ E.			

Right-Hand Page.

	25° 39′ 30″	124° 16′ 35″	124° 16′ 33.4″
	334° 20′ 50″	55° 43′ 28.3″	55° 43′ 26.6″
		180° 00′ 03.3″	
Az. 219-220		85° 58′ 23.5″	

The entry, Δ 218-220, shows that 218 was first sighted to, and that the deflection at 219 was to the north, that is, 218 is the left-hand point, and the transit swings to the north and east in reaching 220.

The 124° 16′ 40″ is the first reading of the angle.

At the end of the six accumulations, the verniers both read 25° 39′ 30″, and at the close they both read 00′ 20″. If they had read differently—as 00′ 00″ and 00′ 10″—at the start or any other place, then another column of mean readings would have shown, as, for the foregoing, 00′ 05″. The notes to this point are recorded on the left-hand page of

the notebook and comprise the record of the field work. The rest are on the right-hand page, and constitute the reduction.

As the verniers both read 00 at the start, the 25° 39′ 30″ is carried across as the true reading of the summation of the six repetitions of the angle. Subtracting this from 360° 00′ 20″ gives 334° 20′ 50″ as the true accumulation of the exterior angle. Dividing each by 6, and adding to the first the necessary full revolutions of 360° (in this case 2) in order to give the proper angle 124°, there results the two angles given. The last one is really 180° less than was measured, but, for purposes of reduction, it is the deflection angle which is wanted, which is the above amount (180°) less than the exterior angle. The sum of these two measured angles is 03.3″ in excess of 180°, and shows the instrumental slip or error in pointing. This is divided equally between the two angles, and the deflection angle at 219 is thus determined to be, 55° 43′ 26.6″ to the left, or north.

The computed azimuth from 218 to 219 was 141° 41′ 50.1″, so that, subtracting the deflection at 219 from this, the azimuth, 219-220, is 85° 58′ 23.5″.

This azimuth is not corrected for easting nor for instrumental error. The former depends on the distance from A-204, where the last elongation was taken, and the latter was found to be, at A-222, where the next star hub was set, 24.9″ too large in 19 angles, or 01.3″ per angle. The final true azimuth of this course was found to be 86° 00′ 04″.

Each night the notes of angles measured during the day, together with the distances, were given to the computer, who calculated the eastings and thus corrected the azimuths and made up a table of preliminary locations from R. P. "E" of all points, so that they could be platted on the map. When the run was tied through to Polaris, and the azimuths were corrected for instrumental error, the final latitudes and departures were computed. The preliminary figures were never in error as much as 1 m., or, they were as close as they could be platted. Of course, they were corrected and started right whenever an elongation was taken.

Accuracy of Angular Work.—Table 1 gives the whole line from Roma to the Gulf, and shows the corrections used. The run from A-144 to A-180 was rejected, and the azimuths of the Mexican Section were used, but it is included here to show the actual results of

the season's work. The total number of angles exceeds the courses given for the A line, because several elongations were taken on spur lines to reference points, and these angles were counted in making the correction as well as the angle to each star. Elongations were taken from the points given in Columns 1 and 2.

TABLE 1.—INSTRUMENTAL ERRORS IN PRECISE LINE.

(1) Star.	(2) To star.	(3) Closing error.	(4) Number of angles.	(5) Error per angle.	(6) Remarks.
Z-399.........	Z-408.........	+ 00′ 14.7″	11	01.5″	
Z-408.........	A-1.........	+ 01′ 07.1″	31	02.1″	
A-1.........	R.P-3.........	− 00′ 14.0″	13	01.0″	
R.P-3.........	R.P-7.........	+ 00′ 38.0″	19	02.0″	
R.P-7.........	A-58.........	+ 00′ 21.0″	12	01.8″	
A-58.........	R.P-12.........	− 00′ 39.9″	24	01.7″	
A-78.........	R.P-16.........	+ 00′ 54.0″	32	01.7″	
R.P-16.........	R.P-18.........	+ 00′ 38.2″	19	02.0″	
R.P-18.........	A-144.........	+ 00′ 50.3″	25	02.0″	
A-144.........	A-180.........	+ 01′ 47.7″	37	02.9″	Rejected.
A-180.........	A-204.........	− 00′ 05.7″	26	00.2″	
A-204.........	A-222.........	+ 00′ 24.9″	19	01.3″	
A-222.........	A-246.........	+ 00′ 11.9″	25	00.5″	
A-246.........	A-260.........	+ 00′ 00.4″	15	......	
A-260.........	A-281.........	− 00′ 36.1″	22	01.6″	
A-281.........	A-300.........	+ 00′ 38.0″	20	01.9″	
A-300.........	R.P-48.........	+ 01′ 02.9″	37	01.7″	
A-334.........	A-374.........	+ 01′ 10.0″	41	01.7″	
A-374.........	A-382.........	− 00′ 18.0″	11	01.6″	
		11′ 52.8″	439	01.62″	
	Rejecting A-144 to A-180				
		10′ 05.1″	402	01.51″	

Table 1 shows that the average instrumental error, in turning 439 angles, was 01.62″ per angle, or, rejecting the run from A-144 to A-180, the average was 01.51″ per angle for 402 angles. This does not mean that, with a 6-in. plate, reading to 20″, each angle can be read with certainty to this degree of accuracy, but that the average error can be, and was, brought down to this small amount. It requires great care in handling the instrument and great care in pointing. The weather conditions also affect the work, but the latter was continued in heat and in wind just as long as the rod could be seen or a man could stand at the instrument. The orders were to keep up with the outfit, and every man was trying to make the others do the keeping up.

The signs in Column 3 of Table 1 show the transitman's personal equation. He nearly always had too large an azimuth. Whenever he failed to close within the limit, his azimuth was invariably too large.

The writer does not know that any precise line work similar to this has been done elsewhere. Triangulation has generally been used for the exact location of points. With an expert instrumentman, the limit of 2″ per angle is permissible, but it is extremely difficult for an ordinary man, however careful he may be, to keep within this limit, and it would be advisable to raise it to 3″. In the spring of 1913, the writer had 60 miles of similar work done in the El Paso Valley of the Rio Grande. The same transit was used, but with another man handling it, who was allowed 4″ per angle for instrumental error. His average error on 234 angles divided into 8 runs was 1.85″. His best run showed a closure of 1.13″ per angle for 37 angles and his worst, 3.20″ per angle for 24 angles.

Table 2 gives the sum of the forward chaining distances (from hub to hub) between star points, the sum of the backward chainings, the sums of the differences between adjacent hubs and their arithmetical sums, and the ratio of error deduced from these, as well as the algebraic sums of errors and the resulting ratios, and the algebraic sum of the total errors with its ratio.

Table 2 shows that the line was broken up into nineteen runs with lengths varying from 5 600 to 22 700 m., and averaging 12 600 m.; it also shows that the number of courses into which each run was broken by angle points varies from 10 to 37, the total being 411. When the forward chaining between adjacent hubs was larger than the backward, the difference was called plus, and the reverse was called minus. These were summated and entered in Table 2 as sum of differences, plus, minus, and total, the latter being the arithmetic sum of the plus and minus sums. The ratio of error was found by dividing the distance by the total differences. These ratios run from 1 in 44 000 to 1 in 184 000, and the average for the whole line is 1 in 91 000. Not a single course was omitted, although in checking the work in the office, one was found which exceeded the allowed limit of 1 in 20 000. It was from A-180 to A-181, the distance being 752 m. and the difference, 0.068 m., or 1 in 11 000. All

TABLE 2.—ERRORS IN MEASURING PRECISE LINE.

(1) Station.	(2) To station.	(3) Number of courses.	(4) Forward.	(5) Backward.	(6) Sum of Differences. +	(7) Sum of Differences. −	(8) Sum of Differences. Total.	(9) Ratio: 1 in	(10) Net Difference. +	(11) Net Difference. −	(12) Ratio: 1 in
R.P-"E"	Z-408	10	5 623.401	5 623.504	0.013	0.116	0.129	44 000		0.103	55 000
Z-408	A-1	23	12 926.099	12 926.143	0.089	0.133	0.222	58 000		0.044	294 000
A-1	R.P-3	12	6 947.551	6 947.594	0.022	0.065	0.087	80 000		0.043	162 000
R.P-3	R.P-7	18	11 123.838	11 123.877	0.023	0.062	0.085	131 000		0.039	285 000
R.P-7	A-58	11	8 968.783	8 968.754	0.058	0.029	0.087	103 000	0.029		309 000
A-58	A-78	21	9 199.590	9 199.549	0.061	0.020	0.081	114 000	0.041		224 000
A-78	R.P-16	29	18 524.327	18 524.411	0.056	0.140	0.196	94 000		0.084	221 000
R.P-16	A-121	17	9 871.010	9 870.988	0.043	0.021	0.064	154 000	0.022		449 000
A-121	A-144	23	12 952.137	12 952.231	0.020	0.114	0.134	97 000		0.094	138 000
A-144	A-180	36	20 442.481	20 442.551	0.116	0.186	0.302	68 000		0.070	292 000
A-180	A-204	25	14 102.703	14 102.665	0.124	0.106	0.230	61 000	0.018		783 000
A-204	A-222	18	11 264.993	11 265.187	0.023	0.217	0.240	47 000		0.194	58 000
A-222	A-246	24	11 924.062	11 924.081	0.034	0.053	0.087	137 000		0.019	627 000
A-246	A-260	14	8 063.971	8 064.029	0.010	0.068	0.078	103 000		0.058	139 000
A-260	A-281	21	11 679.774	11 679.791	0.048	0.065	0.113	103 000		0.017	687 000
A-281	A-300	19	11 037.223	11 037.221	0.031	0.029	0.060	184 000	0.002		
A-300	A-334	34	19 760.327	19 760.415	0.035	0.123	0.158	125 000		0.088	225 000
A-334	A-374	37	22 727.641	22 727.605	0.085	0.049	0.134	170 000	0.036		631 000
A-374	R.P-55	14	12 219.289	12 219.270	0.073	0.054	0.127	96 000	0.019		643 000
		411	239 359.180	239 359.866	0.964	1.650	2.614	91 000	0.167	0.853	
									Net ...	0.686	349 000

other measurements checked within the limit. This ratio of 1 in 91 000 is an actual ratio, without any chance for compensating errors, except such as may occur between the individual chain lengths. Each distance between hubs formed a run by itself, and its error of closure is counted.

As a matter of curiosity, there is shown in Table 2 the algebraic sums of the differences of each run, with the resulting ratios and the final algebraic sum (0.686 m. in 239 km.), with its ratio of 1 in 349 000. This is the total net difference between the forward and backward measurements. As the mean is half way between, either line varies only 34 cm. from the mean for the whole distance from Roma to the Gulf. This, however, is interesting only as showing how errors in work carefully done are likely to balance. The true difference between the two measurements is 1 in 91 000, or the variation of either from the mean is 1 in 182 000.

It should be understood that this chaining was done in the regular day's work, with no arrangement for holding the tape still over the tack, as is usually done. Chaining pins were used, instead of hubs with plates and hair lines, as is sometimes the practice.

Rate of Progress.

Between the beginning and end of the survey about 110 working days intervened. Several banco ties were made by the precise line party, probably 10 days being used on that, thus leaving 100 days for precise line work. About one-third of this time was spent by the chainmen in locating line, and several days were also spent in re-chaining distances which did not check within the limit. There remained about 60 days in which a distance of 150 miles was chained twice, or, the rate of chaining was 5 miles per day of from 8 to 9 hours' actual work. The whole work of chaining and recording was done by the two chainmen, with no assistance except, perhaps, an axeman who carried their water canteens, lunch bucket, coats, etc.

As the transitman measured so many of his angles a second, and sometimes a third time, it is hard to give his rate of progress. He probably could measure from 8 to 10 angles in an ordinary day's work. Fifteen angles per day is fast work for an expert.

PRIMARY LEVELS.

It was desirable to have a moderately accurate line of levels from Roma to the Gulf. There was no occasion for a line of what scientists would call precise levels, run with an instrument of great sensitiveness, shaded from the sun, with three wires to be read, two rods, and rodmen with levels on their rods, etc., etc. It was decided to obtain the best possible results from an engineer's ordinary 18-in. Wye-level and a target rod held on nail heads or round-topped pegs, waved on each reading, and read to millimeters. There was nothing unusual about this level work. The line was run in the ordinary way, that is, a run forward for about a mile was made and checked back. Then the mean height of the point ahead was computed and, using it, another mile was run, etc. The limit of error allowed between benches was

$$0.01 \text{ m. } \sqrt{\text{distance, in miles, between benches.}}$$

Table 3 shows the results of this leveling.

TABLE 3.—CLOSING ERRORS IN PRIMARY LEVELS.

(1) From:	(2) To:	(3) Closing error, in meters.	(4) Distance, in miles.	(5) $\sqrt{\text{Distance.}}$	(6) Coefficient.
R.P-"E"......	R.P-2........	−0.006	13.3	3.65	0.0016
" 2	" 7........	−0.006	9.4	3.07	0.0020
" 7	" 13........	+0.009	15.5	3.94	0.0023
" 13	" 18........	−0.022	13.3	3.65	0.0060
" 18	" 22........	−0.030	11.4	3.38	0.0090
" 22	" 28........	+0.009	18.1	4.25	0.0021
" 28	" 34........	−0.011	13.8	3.71	0.0030
" 34	" 39........	−0.031	13.3	3.65	0.0085
" 39	" 45........	+0.007	11.9	3.45	0.0020
" 45	" 50........	−0.043	13.8	3.71	0.0116
" 50	" 55........	−0.016	15.5	3.94	0.0041
Totals........................		−0.140	149.3	12.22	0.0114

Remembering that the allowable coefficient of error is 0.0100, an examination of Column 6 shows that all the separate runs excepting one, were within the required limit, but that the total was a little outside the limit. The net error was — 0.140 m., the allowable would have been 0.122, or, the line as a whole exceeded the allowed limit by 18 mm.

It will be noted, however, that other combinations can be made which will show larger errors. Table 4 was compiled to show these,

the quantities being taken from Table 3. Column 6 shows that two runs, namely, from R.P-13 to R.P-22 and from R.P-45 to R.P-55, exceeded the limit, the first by 2 mm. and the second by 5 mm.

TABLE 4.—CLOSING ERRORS IN PRIMARY LEVELS.

(1) From:	(2) To:	(3) Closing error, in meters.	(4) Distance, in miles.	(5) $\sqrt{\text{Distance}}$.	(6) Coefficient.
R.P-"E"......	R.P-7........	−0.012	22.7	4.76	0.0025
" 7	" 13........	+0.009	15.5	3.94	0.0023
" 13	" 22........	−0.052	24.7	4.97	0.0105
" 22	" 28........	+0.009	18.1	4.25	0.0021
" 28	" 39........	−0.042	27.1	5.21	0.0081
" 39	" 45........	+0.007	11.9	3.45	0.0020
" 45	" 55........	−0.059	29.3	5.41	0.0109

Table 5 shows the line broken into four approximately equal parts, and all except the last, which is from R.P-45 to R.P-55, are within the limit.

TABLE 5.

(1) From:	(2) To:	(3) Closing error, in meters.	(4) Distance, in miles.	(5) $\sqrt{\text{Distance}}$.	(6) Coefficient.
R.P.-"E"......	R.P.-13........	−0.003	38.2	6.18	0.0005
" 13	" 28........	−0.043	42.8	6.54	0.0066
" 28	" 45........	−0.035	39.0	6.24	0.0056
" 45	" 55........	−0.059	29.3	5.41	0.0109

It is hardly fair to this work to compare it with precise level work, because the latter is done with so much greater care. The following comparison, however, is given.

Johnson's "Surveying" gives the following limits of error—all in meters and kilometers. The equivalent of

0.01 m. $\sqrt{\text{distance in miles}}$ is 0.008 m. $\sqrt{\text{distance in kilometers}}$.

U. S. Coast and Geodetic Survey... $0.004\sqrt{K}$
U. S. Lake Survey................ $0.010\sqrt{K}$
Mississippi River Survey........... $0.005\sqrt{K}$ or $0.003\sqrt{2K}$.

It will be seen that this line of levels meets the requirements of the Lake Survey, but not those of the other two. Although the difference between a coefficient of 8 mm. and one of 4 or 5 mm. is small, the indicated precision is much greater with the smaller coefficients, and necessitates a finer instrument and more delicate manipulation.

In a paper entitled "Surveying,"* by Officers of the United States Geological Survey, it is stated that when duplicate precise lines are run, as in the case under consideration, the error allowed by the U. S. Geological Survey is 0.02 ft. $\sqrt{2D}$ in miles. This coefficient equals 0.007 m.

In Table 6 this is applied to the three worst runs, namely, R.P-13-22, R.P-45-55 and R.P-"E"-55.

TABLE 6.—COMPARISON IN PRIMARY LEVELS.

(1) From:	(2) To:	(3) Closing error, in meters.	(4) Twice the distance.	(5) $\sqrt{2D}$	(6) Coefficient.
R.P-13........	R P-22.......	—0.052	49.4	7.03	0.0074
" 45........	" 55.......	—0.059	58.6	7.65	0.0077
" "E"......	" 55.......	—0.140	298.6	17.28	0.0081

Table 6 shows that the work nearly met the requirements of the Geological Survey for precise levels, run with high-grade instruments, umbrellas, etc.

Rate of Progress.—Including spur lines to reference points away from the precise line, the primary levels covered a double run of about 155 miles, or 310 miles of single line. The leveler worked half the time on this line and half the time on the river. Two or three days were used in going back to check out an error, etc., so that, there having been 110 working days during the survey, 51 or 52 days were devoted to the primary leveling. The rate of progress is seen to have been 6 miles of single line per day.

STADIA LINES.

Thesis.—It is the writer's belief that too many refinements are usually applied to stadia work, and that they give to it a factitious value without adding anything to the accuracy of the results. Thus, rod levels are used to keep the rods plumb; the stadia interval is determined with great nicety for each instrument and each observer; a rating table is made up which involves $f + c$ and is usually carried to centimeters; stadia readings are reduced by this table, thus running all distances into centimeters and giving to the work an appearance of accuracy which does not exist.

* *Transactions*, Am. Soc. C. E., Vol. LIV, Part B, p. 426.

Believing that these refinements are a waste of time, with no corresponding valuable result, but that steady, careful instrument work, disregarding $f + c$, and, in case the stadia interval checks out close to unity on a measured base, using stadia readings direct, without reduction tables, would give just as good results, all the stadia work on this survey was done in this way. The following discussion shows the results.

The principal part of the survey work was the taking of topography by stadia. Three parties were on this portion of the survey from start to finish. One was charged with the river and as much of the adjoining country as could be reached conveniently. The second devoted its time to the Texas side of the river and the third to the Mexican side. The work was platted on the map as soon as possible after being taken, so that running track was kept of the areas not covered. They were examined, and, if anything of importance was found, a line was run to it. About 375 sq. miles of country were covered by the three parties. Altogether, 37 000 shots were taken, locating, by azimuth, distance, and elevation, that many points on the ground. In addition to these, some 9 000 shots were taken in former surveys of bancos, which were platted on the map, so that 46 000 points were located by stadia, or an average of 123 shots (including transit points) per sq. mile. There were 4 900 hubs set, or 13 per sq. mile.

Each stadia party consisted of a topographer, who handled the transit, recorded the notes, and made his sketches; an American rodman who usually kept ahead, choosing places for transit points, and keeping an eye on the stadia men; a rear flagman, two or three stadia men, and as many axemen as were needed. On the river line, a skiffman was also used. The native labor of the country was used for flag, stadia and axemen. Few could speak English, although many of them were born in the United States, as had been their ancestors for generations.

The instruments consisted of a transit and three or four stadia rods. No. 2 transits were used, one reading to 20″ and two to 30″. All had fixed stadia wires which were set by the makers to subtend 1 m. in a distance of 100 m. Careful tests on base lines measured with a steel tape, showed that the setting of the wires was practically

exact. The writer considers adjustable stadia wires a "delusion and a snare," and would not use them under any circumstances.

The stadia rods were of white or sugar pine, with an iron shoe on the bottom, and were of the following dimensions: 4.55 m. (14 ft. 11 in.) long; bottom, 9 by 2.5 cm. (3½ by 1 in.); and top 7.3 by 1.6 cm. (2⅞ by ⅝ in.). They were graduated and painted as shown by Fig. 1. The figures were 4 cm. high and were centered over the decimeter marks. Notches were painted to mark the decimeter at the ones and sevens. Red was used to mark the full meters and one-half of the diamond at the half meters was also red. As the latter color might become indistinguishable from black at long distances, when the light was bad, the form of the diamond was varied at the meter and half-meter, so that they might be located by their shape. In ordinary weather and light these rods could be read with certainty to centimeters at a distance of from 200 to 300 m. Beyond that distance the readings were uncertain.

FIG. 1.

As it was formerly the rule to graduate stadia rods to fit each instrument, it should be noted that these rods were graduated to meters and were interchangeable between the different parties.

The face of this rod was so badly exposed that the figures wore off quickly, and it was also limber and hard to keep from warping. It would be greatly improved by screwing to the edges ¼-in. strips of hard wood which would project ⅛ in. beyond the face.

Programme of Work.—Starting from a point, the location of which was known and where an accurate azimuth could be had, that is, from "A" or "B" hubs, or from banco corners, or, for want of something better, from stadia hubs in a closed traverse, the topographer ran a traverse which closed on another fixed point or on a point in another stadia traverse

which had been closed. Azimuths were read from 0 at the north around to the right, 90° for east, 180° for south, 270° for west, and 360° or 0 for north again. The programme was as follows:

The topographer set up his instrument over the point, read and recorded the height of the axis of the telescope above the top of the hub, computed the back azimuth of the point he wished to back-sight on by adding 180° to its forward azimuth, which had been read to the limit of the instrument, set his A vernier to read this azimuth, and back-sighted, with his telescope erect, on the edge of a stadia rod which was being centered as nearly as possible over the tack in the back hub. He then signaled to the rear flagman, who turned the face of the rod to the instrument. The topographer then read and recorded the level reading to the back-sight, and the distance, unclamped his plate, and was ready to take topography. The river man started with 0 for his first hub, and gave each shot a consecutive number. He usually had three stadia rods, sometimes four, working, and kept them all going. When a rod came up he turned his instrument on it, read the level (the telescope was kept level all the time if possible), turned the micrometer screw until the lower wire caught a full meter mark, read the upper wire, mentally deducting the reading of the lower wire, turned the micrometer back until the middle wire read the level reading, so that the telescope was again level, entered in his book the level and stadia readings, waved the stadia man all right, which released him, and then read and recorded his azimuth. The level reading was taken to centimeters, the stadia to centimeters, and the azimuth to the nearest 10′, except on very long sights, when it was read to 5′. For convenience in platting, one side of the river was given from six to eight consecutive numbers, and then the other side the same, but the readings were taken whenever a rod came up.

When the readings were all taken as far as the transitman could see, the rodman located a hub and put a tack in it, signaling for "point". The topographer set his vernier back to the back azimuth, noted whether or not the transit had settled, re-leveled if it had, and sighted to the back rod, which, as before, was being held over the tack, and then, turning ahead, read and recorded the azimuth to the new hub. The head rodman then turned his rod, and the level and stadia were read, and recorded. In this way levels and distances

were read twice on all turning points—once forward and once backward.

The stadia was read direct, that is, the lower wire was placed, if possible, on an even meter mark, and the intercept between it and the upper wire was read. Sometimes it is specified that, on turning points, the lower, middle, and upper wires shall be read and recorded, and then the intercept computed. It is claimed that this eliminates the danger of errors. This is quite true, but it also eliminates time to such an extent that it retards the progress of the party.

Page of River Book.

From:	To:	Azimuth.	Corrected Az.	Stadia.	H. I.	Rod.	Elev.
	Etc.	Etc.					
4 406 ⊙	4 393 ⊙	238° 30′ 30″	....	2.29	22.09	2.79	20.69
	4 406 ⊙	58° 30′ 30″	58° 27′ 30″	2.29		0.15	1.40
	Cor. 2 ⊙	54° 37′	....	1.985		H. I. + 0° 29′	20.0
	5	98° 10′	....	1.75		0.80	20.0
	4	52° 20′	....	1.65		1.00	19.8
	3	115° 10′	....	1.28		0.60	20.2
	2	64° 40′	....	2.30		H. I. — 0° 59′	15.3
	1	76° 20′	....	1.00		H. I. — 2° 18′	15.3
	4 400	54° 50′	....	1.12		1.00	19.8
	99	170° 00′	....	2.40		1.10	19.7
	98	41° 50′	....	1.18		0.80	20.0
	97	155° 10′	....	0.35		H. I. — 6° 54′	15.1
	96	154° 50′	....	1.17		0.60	20.2
	95	340° 00′	....	0.40		2.80	18.0
	94	340° 10′	....	0.18		4.00 + 2° 20′	20.0
4 393 ⊙	4 382 ⊙	236° 39′	....	3.75	20.80	0.62	19.28
	4 393 ⊙	56° 39′	56° 36′	3.75		2.45	1.52

(From River Book No. 10, page 19.)

When the above record was made, the topographer went ahead to the new point, made his sketch, set up, and repeated the operations.

The notes were kept in books especially made for this work, with wider pages than usual. The notes ran up the page. On the previous page there is a sample of the notes on the left-hand page of a book. On the right hand were made the sketches and such entries, in the form of remarks, as were deemed necessary. The shot numbers, being between 0 and 5 000, show that this was on the river.

The notes on the right-hand page showed that the "Cor. 2" tied to was Cor. No. 2 of La Palma Banco No. 25. It happened that at this point the azimuths were, for some reason, being corrected — 03′. The elevations underscored thus, 19.28, were those of the hubs, and were obtained by the leveler. The entries in the elevation column enclosed in circles were the height of the telescope above the hub, and the entries in the sixth column were the height of instrument above datum. Of course, the notes and sketch on the right-hand page showed where all these readings were taken. This was an ordinary set-up for the river, as regards number of shots, but, on the side lines, fewer shots were taken from each hub, and sometimes brush had to be cut for each shot, so that progress was slow.

Every night the stadia hubs set during the day were platted on the map, and if a line ran to a closure and closed within a reasonable limit, the line was adjusted graphically. The error allowed depended on the length of the traverse, but roughly, was about 2 m. per km. If the error was more than that, the line was usually re-run in the field. If an error of azimuth of more than 5′ developed in the field on closure, the line was re-run. Probably a dozen re-runnings were made during the winter's work, which shows that the topographers were careful with their work.

Use and Abuse of Stadia.—The use of stadia in taking topography has come into vogue in comparatively recent years. It was used extensively on the re-survey of the United States-Mexico boundary west of El Paso, in 1892 and 1893. All the topography which the present Boundary Commission has taken has been with the stadia. It was formerly the custom to use rod levels on the rods, to determine and frequently to check a "stadia interval" for each instrument and each man, to take into account the focal distance plus the distance of the object glass from the axis of the telescope ($f + c$), which, in the small transits used on this survey, is about 30 cm. and to compute distances from a stadia reduction table made for each instrument, and frequently car-

ried to centimeters. Such tables were used by this Commission on the 1897-98 re-survey of the lower river.

As the nearest that the stadia interval can usually be read, at distances of about 200 m. or more, is to the centimeter, which means a meter in distance, it has always seemed to the writer that too great refinements had been attempted. It purported to give to stadia work a degree of accuracy which did not exist, and thus added to its cost without any real gain, as well as giving it a factitious value. All these refinements use up time; the purpose of taking topography is usually to get all one can in the shortest possible time, and to have it so accurate that errors would not be noticeable on the map—in this case 1 in 5 000—which is to be made.

On this work, each man checked his stadia interval on a chained base at the beginning and ending of the work, and one topographer, who seemed to get his distances too short, checked his several times. No material variation from the ratio of 1 in 100 between stadia intercept and distance could be detected. Therefore, the reduction tables were dispensed with, as well as rod levels, and the topographer read his rod as far as he could see it, regardless of whether or not the lower wire cut the rod near the ground. Work was not stopped on account of the "boiling" of the air caused by heat, nor on account of wind, as long as a man could hold a rod. The following analysis of the results will show whether or not it was wise to dispense with these refinements. Of course, if the stadia wires of an instrument do not intercept 1 m. at 100 m., a stadia reduction table must, perforce, be used until the instrument can be sent to the makers and adjusted. The stadia wires should always be "fixed", not "adjustable".

Within its proper sphere, the stadia is unsurpassed in the taking of topography. In any work where a variation of 1 or 2 m. in the relative location of points near together, or 5 or 6 m. in that of those which are material distances apart, can be tolerated, the stadia offers a most rapid and handy method of work; but, where exact, or nearly exact, location of points must be had, as in the better class of land surveys and in all town and city surveys, it is too inaccurate for satisfactory use.

Accuracy of Stadia Work.—After leaving the field, it was decided to compute and balance all the stadia traverses. This gave an opportunity to determine the accuracy of the stadia work. The work

naturally divided itself into three parts, the river line, the Texas topography, and the Mexican topography, but various considerations induced a division of the work into four parts, as follows:

First. All the river work except six traverses: This was all done by an expert instrumentman, with former experience in stadia work. The sights were usually open ones, many being across the water. The distances between hubs were fairly uniform, and were long, as were the traverses, so that errors would compensate. The best results should be expected from this work.

Second. All the Texas traverses, together with two river traverses which were run by the Texas topographer: He was a good instrumentman, without previous experience with stadia; he was working in brush, where he had many bad sights and short distances, and many of his traverses were short. His work could not be expected to show as good results as did that on the river.

Third. The Mexican lines down to San Miguel, about 35 miles below Roma, together with two river traverses near Roma: All traverses were run by green men, and were in brush. All the Mexico lines started from and closed on the river line, so that some discrepancy of closure may be chargeable to it.

Fourth. All the Mexico traverses below San Miguel, together with two river traverses near the Gulf which were run by the Mexico topographer: He was a fairly good instrumentman (with eyes which were slightly defective), without former experience in stadia, and was working the greater part of the time in brush, where sights were likely to be poor and short. Some of the traverses were long, and thus gave a chance for the compensating of errors. Many started from, or closed on, the river line, thus including its error.

TABLE 7.—ERRORS OF CLOSURE AND OF AZIMUTH.

(1) Section.	(2) Number of lines.	(3) Total length.	(4) Number of courses.	(5) Mean length.	(6) Linear error of closure.	(7) Ratio: 1 in	(8) Azimuth error.	(9) Error per angle.
River....	53	395 160	1 140	347	245.2	1 611	70′ 30″	3.7″ (1 140)
Texas....	75	312 721	1 246	251	396.7	788	120′ 12″	7.0″ (1 040)
Upper Mexico.	15	104 742	521	201	130.5	803	13′ 55″	2.3″ (365)
Lower Mexico.	43	238 490	892	267	400.2	596	59′ 45″	4.2″ (847)
Total...	186	1 051 113	3 799	277	1 172.6	896	264′ 22″	4.7″ (3 392)

Although several lines showed evidence of blunders, probably of 10 m. each, in stadia readings, they will all be included in the first analysis, and then they will be dropped and the remaining work analyzed.

Table 7 includes all the stadia lines which were run to a closure.

Some of the lines in Table 7 were run to a distance tie, but not to an azimuth closure. They were as follows:

River section.............	None.	
Texas section.............	14 lines	206 courses.
Upper Mexico section....	5 "	156 "
Lower Mexico section.....	2 "	45 "
Total.................	21 lines	407 courses.

The azimuth errors in Column 9 are calculated with these courses deducted from the totals. The figures in Table 7 in parentheses show the number of courses used in getting the angular error. This table also shows that there were run and computed 186 lines, aggregating 1 051 km. (653 miles) in length, that the average error of closure was, practically, 1 in 900, and the average error of angle reading, where angular closure was had, was less than 5″ per angle.

One river line, one Texas line, and seven Mexico lines indicated "busts" in stadia readings. They are dropped, and Table 8 is made up without them. It gives a fairer idea than Table 7 of the real accuracy of the work. As the mistakes in stadia readings did not affect the azimuth errors, the latter are not repeated in Table 8.

TABLE 8.—Errors of Closure—Omitting Nine Bad Lines.

(1) Section.	(2) Number of traverses.	(3) Total length.	(4) Number of courses.	(5) Mean length.	(6) Linear error of closure.	(7) Ratio: 1 in
River..................	52	383 596	1 112	345	228.0	1 682
Texas..................	74	304 733	1 226	249	377.8	807
Upper Mexico.......	13	87 822	446	197	97.0	905
Lower Mexico.......	38	202 632	768	264	285.8	709
Totals	177	978 783	3 552	276	988.6	990

Table 8 shows that 177 lines, presumably free from "busts", aggregated 979 km. (or 608 miles) in length, and that the average error of closure was a little greater than 1 in 1 000.

Table 9 gives the notes of the nine lines which were omitted from Table 8. It should be understood that these lines were not rejected because their ratio error showed so large, but because the absolute failure to close was so great. Several shorter lines showing larger ratios of error than these, have been left in Table 8.

TABLE 9.—REJECTED LINES.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
						ERRORS OF CLOSURE:			
Section.	From:	To:	Length.	Courses.	Mean length.	Lat.	Dep.	Linear.	Ratio: 1 in
River......	B-279...	R-3 681.	11 564	28	413	15.11 S.	8.29 E.	17.23	671
Texas......	R-619...	R-817...	7 988	20	399	7.37 S.	17.39 E.	18.88	423
Up. Mex...	Z-399...	Z-408 ..	8 782	33	266	9.23 S.	14.19 E.	16.92	519
" " ...	R-00....	R-237...	8 138	42	194	4.67 S.	15.89 W.	16.56	497
Low. Mex..	M-8 151	M-8 336	5 426	20	271	16.91 N.	11.88 W.	20.66	262
" " ..	R-1 997.	R-2 340.	6 601	35	189	2.33 S.	18.56 W.	18.70	254
" " ..	M-9 647	R-96....	6 954	19	366	6.31 S.	17.44 W.	18.55	375
" " ..	R-114...	R-517...	7 686	23	334	21.14 S.	15.51 W.	26.20	293
" " ..	M-7 555	R-1 052.	9 191	27	340	8.95 N.	29.00 E.	30.35	303
		9 lines	72 330	247				184.05	393

Mean length, 8 037.

Table 9 shows that nine lines, aggregating 72 km. (45 miles) in length, have been rejected, because it was apparent that there were one or more mistakes in stadia reading in each. It also shows that their average ratio of error is a little greater than 1 in 400.

In order to determine, if possible, whether or not the different topographers were systematically reading distances short or long, Table 10 was made up. It is assumed that each section had but one series of lines, the general course and total net length of which were as shown in Column 6, which is made up from Columns 4 and 5. The quantities in Columns 4 and 5 were scaled from the map. The sums of the errors of latitude, both north and south, and their net sums, with the proper signs, are given in Columns 7, 8, and 9; the same for departure are given in Columns 10, 11, and 12. Column 13 gives the net linear error of closure, and Column 14 gives the ratio.

Table 10 indicates that the river topographer slightly over-ran on distances, as a general thing; that the Texas topographer balanced remarkably well; but that the Mexico man under-ran quite badly. This confirms the belief that he was inclined to read the stadia too short.

In some places there were two or more side lines parallel to each other, so that the assumption that there was only one series of lines is not exact. There were other disturbing elements which make the results in this table doubtful. It has no especial significance, except that it shows, in a way, the personal equations of the men.

TABLE 10.—GENERAL CLOSURE OF LINES.

(1) Section.	(2) From:	(3) To:	(4) Lat. S.	(5) Dep. E.	(6) Course and distance.
River	10 km. below Roma	9 km. from Gulf	47 000	168 000	S. 74° E. 174 500
Texas	Roma	Gulf	50 000	187 000	S. 75° E. 193 600
Up. Mex	Roma	San Miguel	19 000	43 000	S. 66° E. 47 000
Low. Mex	San Miguel	Gulf	31 000	143 000	S. 78° E. 146 200

(7) Error. N.	(8) Error. S.	(9) Error. Net.	(10) Error. E.	(11) Error. W.	(12) Error. Net.	(13) Net linear.	(14) Ratio: 1 in
64.16	78.88	14.72 S.	120.27	21.81	98.46 E.	S. E. 99.55	1 750
113.14	114.35	1.21 S.	127.78	117.09	10.69 E.	S. E. 10.76	18 000
8.43	27.22	18.79 S.	18.69	52.27	33.58 W.	S. W. 38.48	1 220
133.95	54.40	79.55 N.	22.83	160.46	137.63 W.	N. W. 158.97	920

Tables 11, 12, 13, and 14 show the number and length of the lines in the several sections which had certain ratios of error, and Table 15 shows the same thing for all the 177 accepted lines. These tables also show the poorest and the best ties for each section, and the shortest and longest lines in each section and in the whole.

Table 11 shows that the worst closure on the river was 1 in 408 and the best, 1 in 18 042; that the length of the shortest traverse was 1 889 m. and the longest, 17 897 m., the average being 7 377 m.; that less than 4% of the work was poorer than 1 in 800, that nearly half (48%) was between 1 in 1 000 and 1 in 2 000, and that almost 30% was better than 1 in 3 000. Table 12 shows that the worst closure on the Texas work was 1 in 210 and the best, 1 in 5 542; that the length of the shortest line was 1 329 m. and the longest, 11 320 m., the average

being 4 118 m.; that 3% of the work was poorer than 1 in 400, and that 80% was evenly distributed from 1 in 400 to 1 in 1 500.

Table 13 shows a poorest line of 1 in 285; this was the first traverse of a man without stadia experience. The best line was 1 in 7 596, which was jointly run by two green men. The length of the shortest line was 2 307 m. and the longest, 14 358 m., the average being 6 756 m. Table 13 also shows that 5% was poorer than 1 in 400 and nearly 60% was between 1 in 600 and 1 in 1 000.

TABLE 11.—Ratios of Errors of River Lines.

(1) Ratio: 1 in	(2) Number of lines.	(3) Total length.	(4) Shortest.	(5) Longest.	(6) Mean.	(7) Percentage of whole.
400 to 600...	2	5 230	2 426	2 804	2 615	1.4
600 to 800...	2	8 668	1 889	6 779	4 334	2.3
800 to 1 000...	5	29 754	1 965	11 154	5 951	7.7
1 000 to 1 500...	15	115 307	3 349	13 762	7 687	30.0
1 500 to 2 000...	9	69 659	3 336	13 592	7 740	18.2
2 000 to 2 500...	4	30 899	3 944	12 670	7 725	8.1
2 500 to 3 000...	1	11 911	.....	......	11 911	3.1
3 000 to 4 000...	8	59 264	4 146	14 678	7 408	15.4
More than 4 000...	6	52 904	5 741	17 897	8 817	13.8
Totals.........	52	383 596	.....	......	7 377	100.0
1 in 408.........	1	2 426	Poorest.			
1 in 18 042.........	1	6 856	Best.			
1 in 780.........	1	1 889	Shortest.			
1 in 8 773.........	1	17 897	Longest.			

TABLE 12.—Ratios of Errors of Texas Lines.

(1) Ratio: 1 in	(2) Number of lines.	(3) Total length.	(4) Shortest.	(5) Longest.	(6) Mean.	(7) Percentage of whole.
Less than 400....	3	8 939	1 996	3 491	2 980	2.9
400 to 600....	17	58 050	1 329	5 764	3 297	19.1
600 to 800....	18	60 344	1 657	6 756	3 485	19.8
800 to 1 000....	11	62 114	2 475	11 320	5 647	20.4
1 000 to 1 500....	13	64 216	1 550	8 093	4 940	21.1
1 500 to 2 000....	4	15 709	2 959	5 097	3 927	5.2
2 000 to 2 500....	4	18 777	2 827	8 163	4 694	6.2
2 500 to 3 000....	1	5 920	......	......	5 920	1.9
3 000 to 4 000....	2	6 064	2 433	3 631	3 032	2.0
More than 4 000....	1	4 600	......	......	4 600	1.4
Totals.........	74	304 733	......	......	4 118	100.0
1 in 210..........	1	1 996	Poorest.			
1 in 5 542	1	4 600	Best.			
1 in 481..........	1	1 329	Shortest.			
1 in 979..........	1	11 320	Longest.			

TABLE 13.—RATIOS OF ERRORS OF UPPER MEXICO LINES.

(1) Ratio: 1 in	(2) Number of lines.	(3) Total length.	(4) Shortest.	(5) Longest.	(6) Mean.	(7) Percentage of whole.
Less than 400...	1	4 325	......	......	4 325	4.9
600 to 800...	4	24 462	2 307	8 184	6 115	27.9
800 to 1 000...	3	25 873	4 801	14 358	8 624	29.5
1 000 to 1 500...	1	8 620	......	......	8 620	9.8
1 500 to 2 000...	2	10 645	5 292	5 353	5 322	12.1
3 000 to 4 000...	1	5 845	......	......	5 845	6.6
More than 4 000...	1	8 052	......	......	8 052	9.2
Totals.........	13	87 822	......	......	6 756	100.0
1 in 285..........	1	4 325	Poorest.			
1 in 7 596..........	1	8 052	Best.			
1 in 651..........	1	2 307	Shortest.			
1 in 914..........	1	14 358	Longest			

TABLE 14.—RATIOS OF ERRORS OF LOWER MEXICO LINES.

(1) Ratio: 1 in	(2) Number of lines.	(3) Total length.	(4) Shortest.	(5) Longest.	(6) Mean.	(7) Percentage of whole.
Less than 400...	5	17 484	2 239	6 806	3 497	8.6
400 to 600...	11	59 418	2 421	8 744	5 402	29.3
600 to 800...	8	42 496	3 048	7 484	5 312	21.0
800 to 1 000...	6	38 833	3 073	10 201	6 472	19.2
1 000 to 1 500...	1	5 371	......	......	5 371	2.6
1 500 to 2 000...	3	13 095	2 917	6 449	4 365	6.5
2 000 to 2 500...	1	3 395	......	......	3 395	1.7
2 500 to 3 000...	1	8 540	......	......	8 540	4.2
3 000 to 4 000...	1	7 879	......	......	7 879	3.9
More than 4 000...	1	6 121	......	......	6 121	3.0
Totals.........	38	202 632	......	......	5 332	100.0
1 in 285..........	1	2 844	Poorest.			
1 in 4 081..........	1	6 121	Best.			
1 in 331..........	1	2 239	Shortest.			
1 in 995..........	1	10 201	Longest.			

Table 14 shows that the poorest line on the Mexico work was 1 in 285 and the best, 1 in 4 081; that the length of the shortest traverse was 2 239 m. and the longest, 10 201 m., the mean being 5 332 m.; that nearly 9% was poorer than 1 in 400, and that 70% was about equally distributed between 1 in 400 and 1 in 1 000.

Table 15 shows that the poorest line was 1 in 210, it having been run in Texas; that the best was 1 in 18 042, it having been run on the river; that the length of the shortest line was 1 329 m., it having

been run in Texas, and the longest, 17 897 m., it having been run on the river; that the average was 5 530 m.; that 3% of the work was poorer than 1 in 400; that 75% was about equally distributed between 1 in 400 and 1 in 2 000, with half of this portion between 1 in 800 and 1 in 1 500, and that 15% was better than 1 in 3 000.

TABLE 15.—Ratios of Errors of all Accepted Stadia Lines.

(1) Error: 1 in	(2) Number of lines.	(3) Total length.	(4) Shortest.	(5) Longest.	(6) Mean.	(7) Percentage of whole.
Less than 400...	9	30 748	1 996	6 806	3 416	3.1
400 to 600...	30	122 698	1 829	8 744	4 090	12.5
600 to 800...	32	135 970	1 657	8 184	4 249	13.9
800 to 1 000...	25	156 574	1 965	14 358	6 263	16.0
1 000 to 1 500...	30	193 514	1 550	13 762	6 450	19.8
1 500 to 2 000...	18	109 108	2 917	13 952	6 062	11.2
2 000 to 2 500...	9	53 071	2 827	12 670	5 897	5.4
2 500 to 3 000...	3	26 371	5 920	11 911	8 790	2.7
3 000 to 4 000...	12	79 052	2 433	14 687	6 588	8.1
More than 4 000...	9	71 677	4 600	17 897	7 964	7.3
Totals.........	177	978 783	......	......	5 530	100.0
1 in 210.........	1	1 996	Poorest—Texas.			
1 in 18 042.........	1	6 856	Best—River.			
1 in 481.........	1	1 829	Shortest—Texas.			
1 in 8 773.........	1	17 897	Longest—River.			

Table 15 also indicates, in a general way, that the shorter the line, the poorer the closure is likely to be. This is shown more plainly in Table 11, and bears out the statement that the stadia errors, where the work is skilfully done, tend to compensate each other.

Comparison with Other Stadia Work.—Aside from this survey, the most extensive system of stadia lines with which the writer is familiar was that run by the United States Section of the Barlow-Blanco Commission while retracing the International Boundary from El Paso to San Diego in 1892 and 1893.

Major Barlow states* that, in all, 1 692 miles of stadia lines were run. These were of two classes: First, "Main Lines", namely, straight lines run along the tangents or boundary lines, and consequently free from azimuth errors. Second, "Side Lines", namely, ordinary stadia lines in which both angles and distances were determined. Of the first, 675 miles (1 085 km.) were run, of which five sections, aggregating 125 miles, were checked by triangulation, with an average error

* Personal report on this re-survey, by Major Barlow, pp. 156-157.

in distance of 1 in 1 218. Of lines of the second class, 1 017 miles were run, of which 118 lines, aggregating 514 miles (827 km.), or one-half of the whole, were closed on points on the main line, with an average error in distances of 1 in 752. Of these lines 25% showed an error in closing poorer that 1 in 500; in 31% the error was from 1 in 500 to 1 in 1 000; in 30% the error was from 1 in 1 000 to 1 in 2 000, and 14% of the lines were better than 1 in 2 000.

It will be noted that only a portion of these side lines was controlled by triangulation through the main line. How large a portion Major Barlow does not state. The remaining lines simply closed on the stadia "main line" with its probable error of 1 in 1 200. If these 118 lines were distributed uniformly over the whole area, then only $\frac{125}{675}$, or 22 of them, were controlled by triangulation, as those on the Rio Grande were controlled by the precise line either directly, or through another stadia line. Therefore, on account of involving the error of the stadia "main line", the foregoing closing ratio of 1 in 752 is uncertain; it may be larger or it may be smaller than given.

It should be stated that all the river lines started and closed on precise points; that all the Texas lines, except 10 or 12, started and closed in the same way, and these last were controlled by stadia lines which had been adjusted; and that nearly all the Mexico lines tied in to river traverses, which were all controlled. The rule adopted was: first to balance the traverses which closed on precise points, and then, assuming that all points on these were fixed, to balance the lines which ran from them.

Johnson* gives a table (Table 16) showing the particulars of Major Barlow's 118 lines which were run to a closure. Johnson's last column gives the average azimuth error on closing per kilometer of line run. This has been changed to "Azimuth error per angle read", so that the result may be compared with those of Table 7.

Table 16 shows that the approximate length of the 118 side lines which were run to a closure on the "main", or boundary, stadia line was 7 000 m., or a total of 826.5 km. (513.7 miles) and 18 courses per line; that, assuming the "main" line to be correct—which it was not (see Table 17)—these side lines closed on it with an average error of 1 in 750 in distance and of nearly 23″ per angle turned. These last

* "Theory and Practice of Surveying," by J. B. Johnson, 16th Edition, 1906, p. 280*a*.

two may be compared with the writer's closures (see Table 7) of 1 in 900 for distance and less than 5″ per angle turned, on 186 lines aggregating 1 051 km., or, rejecting 9 lines (see Table 8), a closure of 1 in 990 on 177 lines aggregating 979 km.

TABLE 16.—WORK ON INTERNATIONAL BOUNDARY WEST OF EL PASO, 1892–93.

(1) No. of lines.	(2) Total length.	(3) Average length of sights.	(4) Average number of sights per line.	(5) Error of closure : 1 in	(6) Azimuth error per angle read.
29	111 823.2	253.0	15.2	553	29.0″
49	280 706.8	356.7	16.1	782	23.0″
28	290 633.9	437.7	23.7	817	18.8″
12	143 352.0	580.4	20.6	786	21.5″
118.	826 515.9 7 004 = average length of lines.	386.2	18.1	752	22.7″

The writer has had occasion to check with a steel tape three of the distances between monuments on the boundary west of El Paso, and the result of this checking is given in Table 17.

TABLE 17.—CHECKING DISTANCES BETWEEN MONUMENTS.

(1) From Monument:	(2) To Monument:	(3) Stadia (Barlow).	(4) Chained (Follett).	(5) Difference.	(6) Ratio of error: 1 in
84	85	3 983.66	3 977.6	+6.06	657
85	86	4 449.98	4 444.1	+5.88	756
92	93	4 460.52	4 462.85	−2.33	1 922
		12 894.16	12 884.55	14.27	904

The arithmetical sum of the differences is used. The signs only go to show that sometimes the stadia distances over-ran and sometimes fell short.

The stadia distances in Table 17 are taken from Column 2 of the table on page 36 of Part 1 of the Barlow Report, and it is there stated that the distances are those of the United States Section. The distances between Monuments 84 and 85 and Monuments 85 and 86 were measured with a steel tape on level ground by Mr. Ross Allison in February, 1908. Mr. Allison had just completed several months' work on a precise line in the El Paso Valley and, being a very careful man, he naturally took great pains with his chaining. There is no doubt that it checks within 1 in 10 000. The distance between Monu-

ments 92 and 93 was measured on August 22d, 1906, by Mr. Zayas and the writer. The ground was rolling. The chaining was carefully done, and probably will check within 1 in 5 000 or 1 in 6 000. Table 17 would indicate that the "main" stadia line of the Barlow Survey may not check any better than his side lines, and, as the latter were controlled by the former, considerable doubt is thrown on the trustworthiness of the results given in Table 16. Certain combinations of errors would give results better than there shown, and others would give worse ones.

As illustrating further the possible errors which may have existed in the Barlow "main" line, attention is called to the distance between Monuments 85 and 86. The table referred to in the Barlow Report gives the United States stadia distance as 4 449.98 m. and the Mexican as 4 437.85 m., a difference between the two of 12.13 m., or 1 in 366. The mean was assumed to be correct. The writer does not know whether it or the original Barlow distance was used in balancing adjacent stadia traverses. Similar differences will be found at various places in the table. A cursory examination discloses twenty places where the United States and Mexican distances between adjacent monuments differ more than 1 in 400 and seventeen more where they differ more than 1 in 500. The worst ratio of differences is between Monuments 185 and 186 where the distances differ 1 in 270 (the two distances being 4 496.74 and 4 513.41 m.). The worst absolute difference is between Monuments 105 and 106, where the difference is 21.5 m. and the ratio of differences is 1 in 301 (the two distances being 6 492.22 and 6 470.68 m.). The first is in broken country and the last is on a level prairie.

Attention is not called to these discrepancies with the intention of criticizing the work of the Barlow-Blanco survey, because the exact distances between monuments was of no particular importance, so long as they were placed on the boundary, but for the purpose of showing the uncertainty of the data in Table 16. This table has become a classic, and it is only right that those who use it should know just how much dependence can be placed on it.

On the Barlow work, great exactitude was attempted in the stadia work by the use of rod levels for holding rods plumb, stadia reduction tables, frequent interval determinations, studies of the boiling of the air caused by heat waves, and of differential refraction, etc., etc. All these things not only consumed valuable time, but gave a fac-

titious value to the results. They naturally engendered the belief that the work was much closer than it really was. It is disappointing, to say the least, to find that distances which are given to centimeters are really from one to several meters in error. No man can read by stadia a distance to tenths of a meter at distances of more than 50 m. He may read to half a meter up to 200 m. or so, but beyond that he is lucky if he gets within a meter of the true distance. The centimeters of the Barlow-Blanco distances come from the use of reduction tables which are computed to centimeters from observations which could, at the best, only go to half meters, or in very short distances, tenths of a meter.

Johnson states* that the results obtained on the U. S. Lake Survey are perhaps a fair average for various conditions. On that survey the errors of closure of 141 meandered lines, averaging 1½ miles each, or 210 miles (338 km.) averaged 1 in 650. The official limit was 1 in 300.

Comparison is invited between Tables 7 and 8 and Table 16. The physical conditions under which the two sets of lines were run were about a "stand-off". The Barlow lines were run in a country which was very rough in places, but was practically free from brush, and where the air conditions were generally fair, except in the mountains, where they were usually good. The lines of Tables 7 and 8 were all in a flat country favorable to good work, but, aside from the river, were in brush and with poor air conditions at times. The three tables indicate that much more depends on the expertness and carefulness of the instrumentman than on rod levels and reduction tables. The grade of the transits was the same on both surveys.

The writer believes that his thesis—that stadia work as usually done is burdened with needless refinements—is sustained by the results of this work when compared with those obtained by the more laborious methods.

Rate of Progress of Stadia Work.—The rate of progress on the stadia lines varied greatly. The fairest way of showing it is to give the number of shots per day. These are as follows:

River topography	......	150 to 200 shots per day.
Texas "	......	100 to 150 " " "
Mexico "	......	90 to 125 " " "

The river party covered from 3 to 5 km. (2 to 3 miles) per day.

* "Theory and Practice of Surveying," p. 280.

UNIT COSTS.

The total cost of the field operations, from October 1st, 1910, to June 30th, 1911, including all transportation charges and repair of outfit, was $22 400.

It is impossible to divide this exactly among the various classes of work. About 75% can be properly distributed. This has been done, and then the remaining 25% has been divided up in about the same proportion as the 75%, and the following results:

Precise line	$4 000
Primary level line	750
Topography	13 150
Ties to bancos, marking bancos, and building monuments	4 500
Total	$22 400

The precise line cost includes half of the computer's time. The topography cost includes the time of two draftsmen and the note caller. The banco ties cost includes half of the computer's time.

The precise line party ran and checked (including lines run to reference points) 155 miles of line, which cost $25 per mile. This seems excessively large. No comparison with similar work can be found.

The level party ran and checked 155 miles of primary levels, which cost $5 per mile. This is reasonable, and compares favorably with the cost of similar work in other places.

The topographers covered 375 sq. miles, at a cost (for the work done on this trip) of $35 per sq. mile. There had been, however, 79 bancos surveyed, where old topography was available. Of course, the river had to be re-surveyed in front of the bancos. The average area of each of these to the center of the old channel, was about 145 acres. The survey extended beyond this, so that each banco survey represented, probably, ½ sq. mile, or 40 sq. miles in all, leaving 335 sq. miles which were covered by this survey, at a cost of $39 per sq. mile. No unit cost can be calculated for the banco ties, etc.

It must be remembered that the topography did not cover the whole ground thoroughly, but consisted in the tracing out of special features, such as the river channel, overflow bank, roads, ranches, lagoons, etc.,

hence the foregoing cost is not properly comparable with contour surveys which cover all the ground with equal thoroughness. Such may cost two or three times as much per square mile as did this and still be economically done.

The Barlow survey covered 1 750 sq. miles, at a cost of $85 500 plus 40% for "General office and Commission expenses", or $119 700—or $68 per sq. mile. The two surveys probably covered the ground with about the same degree of thoroughness, but the physical conditions were much more severe on the Barlow work.

DISCUSSION

William B. Landreth, M. Am. Soc. C. E. (by letter).—The writer, who has had experience on stadia topographical work, some of which he has described,* has read Mr. Follett's paper with much interest. The surveys made in 1900 by the New York State Engineer's Department for the Barge Canal, were of the same general character as those described by Mr. Follett. Mr. Landreth.

The writer had charge of the New York Barge Canal Surveys from Herkimer to near Clyde (114 miles), on the Oswego and Oneida Rivers, and for the Cicero Cut-off.

The base line was measured with a 100-ft. steel tape under uniform tension, corrected for temperature, and the leveling was done by duplicate lines run forward and back, using ordinary Wye-levels and New York or Philadelphia rods, with an allowable error of 0.05 ft. $\sqrt{\text{miles between points}}$. The topography was taken by stadia for 2-ft. contours, and plotted on scales of 1 in 3 000 or 1 in 1 500. Wash-drill borings were made at short intervals along both land and stream lines, and cross-sections of streams and lakes were made by soundings. The totals of the various classes of work performed and the cost thereof are shown in Tables 18 and 19.

TABLE 18.—New York Barge Canal, Middle Division, 1900.
Statement of Field and Map Work, Totals.

Location.	Length of transit line, in miles.	Miles of double level line.	Miles of single level line.	Stadia Work.			Ranges sounded.	Map Work.	
				Square miles.	Acres.	Shots per acre.		Sheets.	Scale.
Erie Canal Line	25.84	25.84	80.80	8.26	5 286.4	3.1	....	36	1—1 500
Oneida River Line	.....	.....	.....	1.69	1 083.5	2.7	118.	4	1—3 000
Seneca River Line	53.27	53.27	141.05	16.23	10 385.3	1.8	722.	28	1—3 000
Cicero Cut-off	13.54	.....	18.90	6.52	4 172.8	1.2	....	10	1—3 000
Onondaga Lake	.....	.....	.....	1.35	863.4	2.2	68.	6	1—3 000
Oswego River Line	.....	2.89	5.78	0.39	249.6	6.4	80.	2	1—3 000
Totals, Middle, Division	92.65	82.00	246.53	34.44	22 041.0	2.9	988.	86	....
Herkimer-Utica Line	12.43	12.43	35.47	2.20	1 410.6	8.0	....	14	1—1 500
Totals, all lines	105.08	94.43	282.00	36.64	23 451.6	....	......	100	....

The character of the country and the cost of taking the topography varied greatly in different localities. Along the Erie Canal, from Herkimer to Grove Spring, the country was mostly cleared farm lands, with a large number of buildings in the villages and cities. The terri-

* *Transactions*, Am. Soc. C. E., Vol. XXXIV, p. 281, and Vol. XLIV, p. 92.

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tory on the Cicero Cut-off, and between the mouth of Crusoe Creek and the western end of the Division, was open, cultivated land, but the Seneca and Oneida Rivers were generally bordered by woods or swamps. Work in the Montezuma Marshes was slow, owing to high reeds and soft ground.

TABLE 19.—Cost of Field Work, Exclusive of Head Office.

Class of work.	Cost per mile.	Cost per square mile.	Cost per mile of stream.
Transit line	\$29.11	....	......
Duplicate levels	25.70	....	......
Single levels	8.82	....	......
Stadia work	....	\$175.45	......
Soundings	....	....	\$26.86

Accuracy and Cost of the Work.—The running error on the transit lines, as determined by Polaris observations, was as follows, observations being taken about 5½ miles apart:

Line "A", Utica to Grove Spring:

Maximum running error.............. 9′ 25″
Minimum " " .... 5′ 56″
Average (of 4) running error.......... 7′ 37″

Line "B", Three Rivers to ⅛ Mile East of Clyde:

Maximum running error.............. 12′ 30″
Minimum " " .... 0′ 00″
Average (of 9) running error.......... 5′ 08″

Line "C", Herkimer to Utica:

Maximum running error.............. 8′ 30″
Minimum " " .... 6′ 00″
Average " " .... 7′ 15″

On the leveling, the limiting value of C in the equation error, $C\sqrt{\text{miles between benches}}$, was 0.050, and the actual values were as follows:

Between Herkimer and East Line of Oneida County:

1.—Between Benches:	Distance, in Miles.
a—Maximum value of C = 0.016	0.40
b—Minimum " " " = 0.001	0.99
c—Average " " " = 0.005	12.56

2.—From Origin:	Miles from Origin.
a—Maximum value of C = 0.0078	4.71
b—Minimum " " " = 0.0000	8.14
c—Average " " " = 0.0027	12.56

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Between East Line of Oneida County and Grove Spring:

1.—Between Benches:

	Distance, in Miles.
a—Maximum value of $C = 0.031$	0.9
b—Minimum " " " $= 0.0034$	6.03
c—Average " " " $= 0.013$	25.74

2.—From Origin:

	Miles from Origin.
a—Maximum value of $C = 0.030$	0.90
b—Minimum " " " $= 0.001$	11.00
c—Average " " " $= 0.008$	25.74

Between Phœnix and Clyde:

1.—Between Benches:

	Distance, in Miles.
a—Maximum value of $C = 0.034$	1.81
b—Minimum " " " $= 0.000$	2.84
c—Average " " " $= 0.009$	51.40

2.—From Origin:

	Miles from Origin.
a—Maximum value of $C = 0.042$	6.77
b—Minimum " " " $= 0.007$	51.40
c—Average " " " $= 0.015$	51.40

About half the levels on the line between Phœnix and Clyde were run over very soft ground and through swamps.

Twenty-three stadia circuits run by Party A give a fair example of the results obtained in the stadia work, as follows:

Average length of circuits	5 214 ft.
" number of stations occupied	7
" running error in azimuth	1′ 21″
" " " " " per angle	11.4″
" error in elevation	0.19 ft.
" " " circuit 1	1 214 ft.
Maximum " " " "	823 "
Minimum " " " "	2 424 "

The cost per acre was 84 cents, for transit line, duplicate levels, stadia work, soundings, map work, superintendence, and purchase of supplies, but exclusive of the cost of borings.

The field work on the Cicero Cut-off, a cross-country line 13.54 miles long, covering 4 700 acres, cost 27 cents per acre, exclusive of cost of instruments, equipments, or borings, but including the reduction of all field notes ready for plotting.

The total cost of borings, including cost of outfit, superintendence, and a storehouse, was $10 672.41 for 14 874 ft. penetrated, or 72 cents per ft.

* * * * * * * * *

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The cost of the map work was $121.60 per sq. mile.

Undoubtedly, the number of stadia shots per square mile is a principal factor in determining the cost of stadia surveys, as shown by Table 20.

TABLE 20.

Survey.	Barge Canal, Middle Division.			
	A.	B.	C.	D.
Area, in square miles	13.00	11.60	4.20	11.20
Shots per square mile	2 217.00	1 210.00	461.00	900.00
Cost " " "	$257.54	$214.69	$76.39	$161.36

Varying conditions of woods, hills, streams, houses, etc., make it very difficult to predict the cost of stadia work, but the foregoing data may serve as a guide to engineers contemplating work of similar magnitude.

In 1901, the writer had charge of running a line of spirit levels to connect the various lines previously run by the U. S. Deep Waterway Commission, or the 1900 Barge Canal Survey, to furnish continuous lines of bench-mark elevations along the proposed Barge Canal between Albany and Buffalo, Syracuse and Oswego, and from Albany to Whitehall at the head of Lake Champlain, and on those portions of the Erie Canal not covered by the former surveys.

His report thereon was published in the 1901 Annual Report of the State Engineer, Edward A. Bond, M. Am. Soc. C. E. In order to make the results available, the following extracts are taken from the State Report:

"Work in the field was begun at the old grist mill bench mark at Greenbush March 1, 1901, and completed to Herkimer June 20th. Check lines between the Barge Canal benches on the Seneca River and the old benches on the Erie Canal were run at Syracuse, Peru, Weedsport and Montezuma between June 20th and July 7th. From July 7th to August 17th a portion of the party was employed in the Albany office working up the results of the field work. A single line of levels was run on the Champlain Canal from Watervliet to Whitehall between August 17th and September 14th, and duplicate lines from New London to Clyde along the Erie Canal between September 16th and December 10th."

* * * * * * * * *

"The party was constituted as follows: recorder, instrument-man, two rodmen, and a bubble tender. The chief of the party acted as recorder, or, instrumentman as the necessities of the case required."

* * * * * * * * *

"Instruments, Rods and Appliances. Mr. Landreth.

"The instrument used was a Gurley 'Y' level, purchased in 1900 for the Barge Canal survey. The dimensions of the instrument were:

"Focal length ....$16\frac{1}{2}$ inches.
Clear aperture of objective.......... $1\frac{1}{4}$ inches.
Magnifying power ....35 diameters.
Value of one division of level bubble (measured) ....7.04 seconds.
Value of one division of level bubble as given by makers............10 seconds."

* * * * * * * *

"The rods used were improved Gurley New York rods having a special target and folding disc plumbing level. The face of the target had a black background with a narrow white band along its median horizontal line. The white bands were one-fourth of an inch wide at the outer edges of the target, narrowing down to one-thirty-second of an inch at the center of the face, and allowed a closer setting of the target than the older form of targets.

"The rods were divided into feet, tenths and hundredths, and were read to thousandths by a vernier on the target. The foot of the rod was a bronze casting terminating in a truncated pyramid one-half an inch square."

* * * * * * * * *

"Steel pins, twelve inches long, one inch square at the top, tapering to a point and having a shoulder three inches long carrying a hardened steel cone were used for turning points. The pin was driven securely in the ground with a mallet, striking on the head, and the rod was held on the hardened steel cone, care being taken not to disturb the pins in any way until all readings were taken. The level was shaded at all times by an umbrella when set up, and a cloth bag when moving from point to point. A canvas wind breaker, ten feet long and five feet high, was stretched between one and one-half inch gas pipes driven firmly into the ground."

Duplicate lines of levels were run forward and backward and the limit of the error of closure of the two runnings was 0.020 ft. $\sqrt{\text{distance in miles between benches}}$, from Albany to Herkimer, and 0.016 ft. $\sqrt{\text{distance in miles between benches}}$, on all other lines along the Erie Canal.

"Procedure of Work.

"Starting from a bench or turning point, the instrumentman paced along the towpath from 200 to 250 feet and set up the level, protecting it by the umbrella and wind shield as occasion required.

"Rodman No. 1 remained at the bench and rodman No. 2, starting at the same bench, paced to and beyond the instrument till he reached a point as many paces beyond the instrument as the instrument was from rod No. 1, at which point he drove the steel pin.

"Having carefully leveled the instrument, the leveler set the target on rod No. 1 as a backsight, and then, avoiding both haste and delay, turned the telescope to rod No. 2, and set the target as a foresight. The

Mr. Landreth.

bubble tender kept the bubble constantly in the middle of the tube by slight pressure of the fingers on the leveling plate of the instrument. To balance errors due to defective vision of the bubble tender or differences in the light on the bubble, the bubble tender moved around the tripod when the telescope was turned.

"The recorder remained with rodman No. 1 until both he and the rodman had read, recorded and checked the rod reading, when he walked rapidly to pin No. 2, checking the paced distances on the way. The recorder then read, recorded, and checked the reading of target No. 2 and signaled 'all right' to the instrumentman, who repeated the signal to rodman No. 1, when they both moved forward. Rodman No. 1, going to rod No. 2, read, recorded, computed and compared results with rodman No. 2 and the recorder, the leveler having at the same time paced up to point No. 2 to check the pacing and then paced past point No. 2 the proper distance and set up the instrument. Rodman No. 1 paced up to the instrument from point No. 2 and then an equal distance beyond it and drove steel pin and set target. Thus this alternation occurred: First set up, rod No. 1 on backsight is set first and rod No. 2 on foresight is set last; on second set up, rod No. 1 on foresight is set first and rod No. 2 on backsight is set last."

* * * * * * * *

"The length of line run each day depended almost entirely on the wind and the condition of the atmosphere, and work was stopped when it was found that three or more readings were necessary in order to obtain two readings within two-thousandths foot of each other. The best results were obtained by sights of from 200 to 225 feet. The progress records for the various portions of the survey are given in Table No. 2.*

TABLE 21.—Barge Canal Levels, 1901.

Location.	Days in field.	Miles of single line.	Miles of finished line.	Miles of single line per day.	Cost, per mile, for field work of finished line.
Greenbush-Herkimer.......	77	343.03	95.43	3.16	$27.70
Grove Spring-Clyde........	57	199.18	74.93	3.49	30.90
Tie Lines....................	4	12.50	5.75	3.12	28.00
Oswego Canal...............	5	16.60	8.00	3.32	19.38
Champlain Canal...........	19½	73.26	.....	3.8	11.12

"Accuracy of the Work.

"Table No. 8† has been prepared to show the differences between the east and west lines of this survey. In that table column 1 gives the serial number of the bench mark; column 2 the distance of the second bench noted in column 1, in miles from Greenbush; columns 3, 4, and 5, the difference between the bench marks as given by the west line, the east line and the mean thereof; column 6 shows the partial excesses obtained by subtracting the difference of elevations as determined by the west line from those determined by the east line;

* Rearranged and reproduced herewith as Table 21.

† Not reproduced in this discussion.

column 7 shows the total excess up to that bench mark, the total excess being the algebraic sum of all of the preceding partial excesses. In columns 6 and 7 the plus sign denotes that the east line is above the west line, and the minus sign the reverse. Columns 8 and 9 give the value of 'C' in the equation error = C $\sqrt{\text{miles}}$ between benches, between successive benches and from the Greenbush bench respectively. Mr. Landreth.

"Dividing the line from Greenbush to Buffalo into circuits according to the individual surveys and taking the values of 'C' from column 8, as calculated between successive bench marks, as being the severest test of the accuracy of the work, we have the following table:*

TABLE 22.—RESULTS OF LEVELS, 1900 AND 1901.

Circuit number.	Length, in miles.	Location.	Allowable value of "C."	Maximum. "C."	Times zero occurred.
1	95.42	Greenbush to Herkimer	0.020	0.016	43
2	12.56	Herkimer-East line Oneida County	0.050	0.016	2
3	25.74	East line Oneida County to Grove Spring	0.050	0.045	0†
4	74.93	Grove Spring to culvert east of Clyde.	0.016	0.016	10
5	56.7	Culvert east of Clyde to Rochester	0.050	0.049	6
6	94.19	Rochester to Buffalo	0.050	0.038	11

† Minimum = 0.001.

Taking by themselves the separate circuits given in Table 22, and comparing their east and west lines, shows their divergence, as in Table 23.

TABLE 23.—DIVERGENCE OF LINES OF CIRCUITS.

Circuit.	Maximum divergence, in feet.	Times zero occurred.
1	0.067	15
2	0.014	4
3	0.050	4
4	0.063	4
5	0.198	2
6	0.135	9

For the line from Albany to Buffalo, 354 miles long, the total divergence was 0.276 — C = 0.014.

"Lines Re-Run.

"The lengths of the lines re-run varied somewhat on the various surveys, owing mainly to their having been run in different seasons of the year, and during the work of 1901, the amounts re-run were: Between Greenbush and Herkimer, 26 per cent.; between Grove Spring

* This refers to Table No. 6 of the original report: the greater part of it is reproduced herewith as Table 22.

Mr. Landreth.

and Clyde, 30 per cent. of the total length of east and west accepted lines."

* * * * * * * * *

"The men employed on the Barge Canal lines were taken from the State Civil Service list, and had no special training in accurate leveling, though the men employed in 1901 nearly all had experience in similar work in 1900.

"The instruments used were the regular engineer's levels with sensitive bubbles, but could in no sense be called 'precise levels', as the term is used in Government reports.

"The results are those obtained by men of average ability and carefulness working under rigid instructions with instruments such as may be obtained from any reputable maker, and it should be distinctly understood that no claim is made that the lines run are 'precise levels' in the technical sense of the term.

"The methods of work were almost identical with the later methods of the U. S. Coast Survey and of the U. S. Geological Survey but the levels used were inferior to the precise levels used by the latter in the optical power of the telescope, in weight and solidity and of a much lower cost. The results are those obtained with an average leveling party working at a good rate.

"Experience gained on the Barge Canal surveys shows the necessity of certain precautions to secure a uniform degree of accuracy. Among them may be cited the following:

"1. Before testing the instrument adjustments it should be set in the shade and allowed to remain a few moments, in order to allow all of its parts to come to the same temperature.

"2. During bright sunlight the line of sights should not be near the ground, or a fence, stone wall or building, to avoid the action of the heat radiated from them.

"3. After the target is set and clamped another careful observation should be made of the contact of the rod with the turning point, the plumbing of the rod and the centering of the instrument bubble before the final acceptance of the target setting.

"4. During windy weather the instrument should not be set up in dry sand or dust, as the vibration of the tripod legs causes fine particles to settle under them, raising the instrument.

"5. After the instrument is leveled the observer and bubble tender should stand near it as little as possible, owing to the effect of the heat of their bodies in changing the temperature of parts of the instrument. They should, as far as possible, place their bodies so that their breath will not be blown upon the instrument.

"The essentials for obtaining good results are: A good instrument with a sensitive bubble, kept in perfect adjustment; equal back sights and fore sights; protection of the instrument from the direct rays of the sun at all times; cessation of work when bad air or wind does not allow two settings of the target on the same point within 0.002 of a foot. The chief of the party should be a careful, patient man, who should early learn when to stop work, and his guide should be accuracy first, speed second."

W. N. BROWN, M. AM. SOC. C. E. (by letter).—Mr. Follett advances certain views concerning the accuracy of the stadia and its limitations. Although they may be true as applied to this special piece of work and the methods therein used, they are certainly far at variance with the writer's experience, and are not applicable to stadia work in general. The author fails to appreciate the accuracy with which stadia measurements are being made. Under the heading "Use and Abuse of Stadia", Mr. Follett states: Mr. Brown.

"In any work where a variation of 1 or 2 m. in the relative location of points near together, or 5 or 6 m. in that of those which are material distances apart, can be tolerated, the stadia offers a most rapid and handy method of work."

If Mr. Follett had said feet, instead of meters, he would have been much nearer the accuracy attainable in stadia measurements. The source of his error is in the type of rod used. His results were probably as good as could be obtained with that rod, whether $(f + c)$ and stadia factor corrections were applied or not. That the rod was not capable of close reading is indicated in his statement: "As the nearest that the stadia interval can usually be read, at distances of about 200 m. or more, is to the centimeter, which means a meter in distance". During the past year the writer has had a number of topographic field parties under his direction on the Topographic Survey of Cincinnati. (Area 100 sq. miles, scale of map, 400 ft. to 1 in.) In carrying out this work approximately 1 650 miles of stadia traverse have been run—in street location and across country—filling in topography. This traverse was controlled by triangulation, so that the accuracy of the stadia measurements was always under observation and determined by each closure on control points. One of the specifications for this work was:

"All horizontal distances between well-defined points must scale correct to within the smallest distance which it is possible to plot on the map. This distance is 1/80 of an inch, representing 5 ft. on the ground."

This did not apply to short distances alone, but to distances between points anywhere on the map. That this condition was met is shown by numerous tests to which the map has been subjected by the Department of Sewerage Investigation, City of Cincinnati, before acceptance of the work from the contractors (the work was done by contract).

On this work, rods 3 in. wide and 12 ft. long, divided to feet, tenths, and hundredths of a foot, were used. It was found that, up to 200 ft., the intercept could be read to the nearest half division on the rod, representing ½ ft. in distance; and, up to 600 ft., the intercept could be read to the nearest division on the rod representing 1 ft. in the distance. The length of sight was kept less than 600 ft., as far as possible, especially on long traverse lines.

Mr. Brown.

When reading distances to the nearest foot, instead of the nearest meter or two, the necessity of applying corrections for stadia factor and $(f + c)$, which often amount to as much as 2 ft. in 100 ft. of measured distance, becomes apparent.

Although the need of rod levels is not so apparent on level ground, it is nearly impossible for a rodman to hold the rod truly vertical on a steep slope, as a very slight inclination either way will change the intercept on the rod one or two hundredths, thus introducing a corresponding error of 1 or 2 ft. It certainly seems advisable to have such levels, so that they may be used when necessary.

It is seldom that one finds an instrument in which the ratio of the distance intercepted on the rod to the distance being measured is exactly 1 to 100. Out of eleven stadia instruments used on the Cincinnati work, three had the wires placed so accurately that this ratio was true up to 800 ft. The eight remaining ones required corrections varying from 0.75 to 3 ft. per 100 ft. of distance. They were a fairly representative group of instruments: Seven were by Bausch and Lomb, three by W. and L. E. Gurley, and one by Young and Sons.

It would seem that the condition of a number of instruments without stadia factor, as found by Mr. Follett, is exceptional. The question naturally arises: Was this not partly due to the fact that his rods were not adapted to close or nearly exact reading, and may not some of his "variation of 1 or 2 m. in the relative location of points near together", be undetected stadia factor crying for recognition?

Far from agreeing with Mr. Follett that the stadia is too inaccurate for satisfactory use in the better class of land surveys and in city and town surveys (meaning topographic surveys; for manifestly it is not applicable in city land surveys where land is so valuable that discrepancies in distance of 0.1 ft. are of serious moment), it is the writer's belief that the stadia can be used without any injury to the accuracy of the results, wherever these results are to be plotted on a scale of 50 ft. to 1 in. or smaller scales. In other words, the errors of plotting on a scale of 50 ft. to 1 in. are just about equal to the errors of stadia measurements when properly carried out, and the necessary corrections for stadia factor, focal length, etc., are applied. Of course, with stadia as well as with any other method, there must be proper primary control.

Relative to cost, Mr. Follett gives \$39 per sq. mile, adding that this "is not properly comparable with contour surveys which cover all the ground with equal thoroughness. Such may cost two or three times as much per square mile as did this and still be economically done". The writer is afraid that this is misleading and will cause many an engineer to underestimate greatly the cost of topographic surveying, if he uses even ten times this cost as a maximum. There are so many conditions affecting the costs of topography that it is dangerous indeed

to hazard even a guess until the individual conditions are very thoroughly examined. The elemental conditions affecting costs are: Mr. Brown.

(*a*) The scale on which the map is to be plotted;

(*b*) The degree of accuracy to be maintained;

(*c*) The physical conditions of the area, affecting cost both of control and topography, such as quantity of brush, steepness of slope, and inaccessibility;

(*d*) The amount of culture, and of traffic interference, the latter being especially important in considering city work.

It is the writer's opinion that topography may cost from $30 to $1 500, or even $2 000, per sq. mile, depending on these varying conditions, and still be most economically executed.

N. T. Blackburn, Assoc. M. Am. Soc. C. E. (by letter).—The writer was introduced to the practical side of the stadia method of surveying immediately after graduation from college, and has had a good deal of experience in its use along the coast of Texas, under the U. S. Engineer Office. Its adaptability and flexibility are so great that he has used it almost entirely, except for hydrographic surveys which take in large areas some distance from shore. Where two transits or sextants are not available for locating lines of soundings by intersection, and the lines do not run too far out from shore, a very good survey can be made by locating points on the lines of soundings with a stadia rod held in the sounding boat. Mr. Blackburn.

The writer agrees with the author as to the use of fixed stadia wires and the disregard of the $f + c$ factor. Where carefully measured base lines are run by this method, taking short distances, when the atmospheric conditions are favorable, and with great care in reading the rod, its use may be advisable. For ordinary work the $f + c$ factor is certainly a useless refinement, particularly on side shots. The writer has never attempted to read the rod closer than the nearest 5 ft. on side shots of any length, and he has never been able to guarantee that his side shots in open country, in the middle of the day, for distances of more than 1 500 ft., were within 15 ft. of the correct distance, on account of the trembling of the air. With ideal weather conditions—a cool, cloudy day with no wind, or very light wind—and short shots, it may be possible to read the rod close enough to allow a correction for $f + c$; but the writer's experience has been that it is generally impossible even to graduate a rod to fit the instrument exactly the same at two different times in the day.

On his first work the writer used the ordinary diamond diagram for the stadia rods, Fig. 2. This marking is plain and easily read except that in the hot sun in open country and at distances of 1 000 ft. or more, the red triangles tend to blur, and the dividing line is hard to distinguish. Moreover, any rod which has the same diagram for each

Mr. Blackburn.

1-ft. interval is confusing to read. The red figures, however, are an advantage in woods or brush, as they show up in contrast to the foliage and undergrowth. A better form of marking is shown by Fig. 3. This was given to the writer by a member of this Society who was connected with the U. S. Reclamation Service at one time, and is understood to have been used by that organization. It will be noted that only every fourth foot is graduated to tenths. One stadia wire is set on a dividing line between foot marks so that the other wire falls within the divided block. Distances can be thus read more rapidly, particularly for long shots, as the length subtended between stadia wires can be read in 400-ft. blocks.

A rib on the back of a rod is useful to prevent warping and to furnish a handle by which to hold it, but it adds to its weight and complicates its use as a line rod when taking azimuth. It has been the writer's experience that stadia rods warp badly unless carefully watched.

The author's tables showing errors of closure for stadia traverses are interesting and instructive. On a survey of Buffalo Bayou, Texas, a base line was run along each bank by transit and stadia, and the two lines were tied across at intervals of from 1 to 3 miles, thus forming a series of closed traverses from which the errors of closure could be computed. About six of these closures were thus made, and it was found that the error varied from 1 in 155 to 1 in 2 200. The distances on the 1 in 155 traverse were afterward chained and the error reduced to 1 in 1 930. The stadia work was done very carefully, and all distances were read forward and back, but no $f + c$ correction was made.

In keeping notes the writer has always numbered consecutively his stadia hubs, and also all side shots from each hub. Then any side shot is located by a double number, first, that of the stadia hub from which it was taken, and then that of the side shot, thus: 15-1, 15-2, etc. A page of notes would appear as shown by Table 24. True south is taken as zero and true north would then be 180 degrees.

FIG. 2.

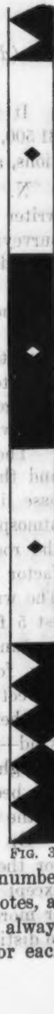


FIG. 3.

On the right page is made a rough sketch of the area surveyed, and the side shots are spotted thereon with their number placed alongside. A note is also made, on the same line as the notes, as to the object on which the side shot was taken. This method has always been clear to the draftsman. As a usual thing, it is better for each

man to plot his own work, but this is not always possible, particularly if it is to be plotted as fast as taken. Where the rodman remains always some distance from the instrumentman, and is able to sketch fairly well, it is usually a good practice to furnish him with a book in which to sketch the topography and locate the points on which he has held the rod. Mr. Blackburn

TABLE 24.

At.	On.	Distance, in feet.	Azimuth.	Magnetic bearing.	Vertical angle.	Elevation.
$\boxed{15}$	$\boxed{14}$	563	12° 20′	S. 4° 30′ W.	....	....
(H I. = 15.4)	$\boxed{16}$	750	220° 49′	N. 32° 50′ E.	....	....
....	1	580	270° 50′	....	5° 10′	....
....	2	650	155° 13′	....	....	....
$\boxed{16}$	$\boxed{15}$	750	40° 49′	S. 32° 50′ W.	....	....
(H I. = 17.2)	$\boxed{17}$	920	187° 15′	N. 0° 45′ W.	....	....
....	1	220	192° 00′	....	....	....

The writer has always used with success the method of observation on Polaris for azimuth given in Johnson's "Surveying."* However, it is plausible that the heat of the light used to read the verniers might affect the instrument, and he is glad to know of this.

During the winter of 1911-12 the writer organized and supervised a topographical survey of the Guadalupe River, Texas, on which methods similar to those described by the author were used, although not so accurately in some particulars. This survey extended over about 38 miles of the river, from a point about 14 miles above the mouth to the Town of Victoria (Mile 52), and covered not only the bed and banks of the river, but an area about ½ mile back on each side. All distances were measured by stadia. A base line was run along the bank of the river, crossing from one side to the other wherever necessary to get the most open country. On this base a double-rodded line of levels was run very carefully. This base line then formed the horizontal and vertical control for the entire survey. The meanders of the river and all topography between the water's edge and the top of the bank were taken directly from this line. The topography back from the river bank was obtained by running random lines from points along the base. These were similar to the banco traverses described by the author. Most of the elevations for topography were obtained by level, running alongside the transit. The transitman

* Page 558 *et seq.*, 15th Edition.

Mr. Blackburn. located the controlling points, and the levelman obtained the elevation, which was immediately given to the transitman and entered in his notes. This method requires more instrumentmen, but it is very rapid and simple, and saves much time in the reduction of stadia notes. Where it would have required several "set ups" of the level to obtain the elevation, resort was occasionally had to vertical angles. The true meridian was obtained from a magnetic base established by the U. S. Coast and Geodetic Survey at Victoria, and was checked at a place about 14 miles below the starting point by direct observation on Polaris by the method previously referred to. The field work of this survey cost about $125 per sq. mile. This high cost was due to the bad weather encountered and to the thoroughness with which the work along the river was done. Much of that work has not been described herein. The cost includes also the purchase of considerable equipment, fitting up a houseboat, etc. The survey was made for the purpose of preparing an estimate and report on the improvement of the river by locks and dams. For the area back of the river bank the cost per square mile was probably less than one-half the figure given.

Mr. Smith. LEONARD S. SMITH, M. AM. SOC. C. E. (by letter).—This paper is a notable contribution to the literature of topographic surveying, and is of special interest to the writer because of its comparisons with the topographical work of the Barlow United States and Mexican Boundary Survey.* In 1892-93 the writer made most of the actual stadia measurements on the tangent or boundary line from El Paso to Yuma, Ariz., besides sharing in the topographic traverses in the same region. In the 20 years which have elapsed since this work was done, he has given a good deal of thought to this subject in connection with both his instruction in topographic surveying at the University of Wisconsin, and also in certain stadia surveys of Wisconsin's most important lakes and rivers, some of which are described later. This work included the running of more than 2 000 miles of stadia traverses, and was secured under a great variety of climatic and other conditions, and for a variety of purposes. This experience has often illustrated the fact that there are many ways in which to make stadia measurements (as well as steel tape measurements), all depending on the character of the work and the accuracy required. Stadia measurements, in fact, should mean many things to all men, not one thing to some men.

The paper includes certain criticisms of the Barlow International Boundary work. The reader must not forget that this field work, as a matter of fact, was completed more than 20 years ago. In what line of engineering endeavor would it not be possible to criticize the

* "Topography on the Survey of the Mexico-United States Boundary," by J. L. Van Ornum, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XXXIV, p. 259.

work of two decades ago, in the light of present knowledge? It would be difficult to show that this work was not the most wisely planned topographic work of its day. The country covered included hundreds of miles of trackless deserts interspersed with scores of mountain ranges, all uninhabited and in most part devoid of water. In the Rio Grande boundary work, such extremes were not present, and, in general, conditions were much more favorable for accurate work. Mr. Smith.

The writer agrees thoroughly with Mr. Follett that adjustable stadia wires are "a delusion and a snare", also that rod levels for keeping the rod plumb are generally unnecessary, but he cannot agree with his statements regarding the general needlessness of an interval determination. It would be a very unimportant piece of work indeed on which the writer would use a steel tape without testing its length as reported by the dealer. The testing of the wire interval of a transit supplies the same information as the testing of the steel tape. So far as concerns the need of either test, it must be admitted that the wire interval is much more likely to change in the field than is the length of a steel tape. The author advocates sending the transit from the field to the manufacturer for readjustment of the interval whenever changes make this necessary.

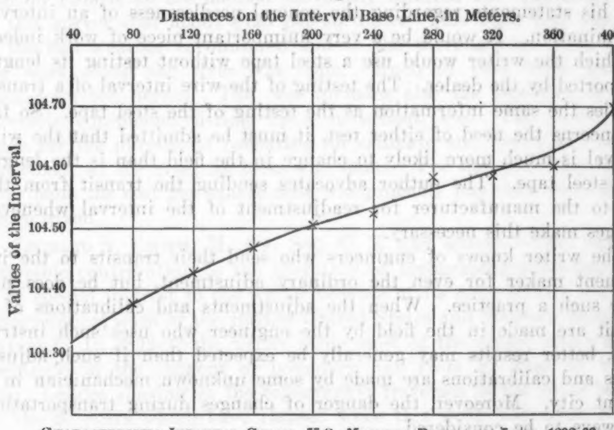
The writer knows of engineers who send their transits to the instrument maker for even the ordinary adjustment, but he does not favor such a practice. When the adjustments and calibrations of a transit are made in the field by the engineer who uses such instrument, better results may generally be expected than if such adjustments and calibrations are made by some unknown mechanician in a distant city. Moreover, the danger of changes during transportation is always to be considered.

A careful examination of the fifty transits belonging to the Surveying Department of the University of Wisconsin, as well as a score of other instruments with which the writer at some time has been familiar, discloses the fact that only rarely has the stadia interval been found to be even approximately 100. In fact, even if the stadia wires were exactly one-one hundredth part of the focal length of the lens in the factory, one could not be certain that they would intercept 1 ft. on the rod for every 100 ft. of distance when used under vastly different circumstances in the field. This is not a matter of theory with the writer, but of frequent and accurate determination. He feels more certain to-day than he did 20 years ago that the general rule regarding the calibration of instruments applies equally well to the stadia interval; namely, that such calibrations, as far as convenient, should be made under the same circumstances as those to be met in the field work. Mr. Follett does not seem to think such precautions likely to result in more accurate work, and quotes certain stadia measurement made by the writer in 1892 to prove his point.

Mr. Smith.

It is now proposed to use these same stadia measurements, together with seventeen others, all checked by triangulation and steel tape measurements, to show that had not less but more care been taken in one important particular, a much higher accuracy would have resulted.

In determining the stadia interval the method was to read the rods every 40 m. on a measured base line 400 m. long. A single determination of the interval was thus secured at each 40-m. point. These values are platted as the ordinates and the distances as abscissas of a curve, which, much generalized, is shown in Fig. 4. The adopted or working value of the interval was the average of all the ten values obtained



CHARACTERISTIC INTERVAL CURVE, U.S.-MEXICAN BOUNDARY LINE, 1892-93.
FIG. 4.

at the various base distances. On account of the systematic differences in these partial intervals, this method would be correct only in case the stadia readings of the later field measurements were distributed approximately over the same range in distance as those of the interval determination. Now, in measuring the boundary line, rod readings at greater distances than 270 m. were not taken. The writer has reviewed the original field books concerned, and has found that had the computations for the stadia interval been restricted to the corresponding distance limit (270 m.), the resulting working interval would have been 104.44 instead of 104.60. This method was not followed at the time the measurements were made because its necessity then was not realized.

When preparing their report, 2 years later (1895), this feature of determining an interval was discussed and approved by the U. S. Commissioners. Table 25 is a record, which was checked by triangulation or steel tape measurements, of the writer's boundary stadia

measurements and taken from page 235 of the U. S. Commissioner's report. A study of this table discloses the fact that the sign of the error of every stadia measurement checked is positive, that is, the distances, as measured by the stadia, were in every case too long, instead of being sometimes too short and sometimes too long, as would have been the case had the interval been free from large systematic error. The accumulated error of the 80 775 m. of stadia measurements was 94 m., or 1 in 858. Mr. Smith.

If these same measurements are computed by the use of $k = 104.44$ —the proper interval obtained, as explained previously—the resulting distances are shown in Column 7, giving the errors as stated in Column 8, and expressed fractionally in Column 9.

It will be noted that the errors of these measurements are of both signs, and that the total accumulated error of the 80 681 m. is only 28.8 m., or 1 in 2 800. It is also interesting to note that the two distances, Monument 84-85 and 85-86, measured by the writer (Nos. 8 and 9 of Table 25), and checked with steel tape by the author (Table 17), when computed by the proper interval factor (104.44), give results in remarkable agreement with the author's chained distances. Thus, the first measurement, Monument 84-85, checks exactly, and the stadia distance, Monument 85-86, checks with an error of only 0.9 m., or 1 in 4 930, instead of 1 in 657 and 756, as given in Table 17. The fact that the interval factor, 104.44, was computed from the actual field notes and published 15 years before the author's steel-tape measurements were made, should prove the entire reliability and independence of the writer's statements and comparisons. This work emphasizes the fact that an accurate stadia interval must be secured in order to prevent large systematic errors. The greatest advantage of stadia measurements—the certain approximate balancing of the accidental errors—is neutralized if large systematic errors are allowed to enter.

The measurements given in Table 25 are less than 10% of the boundary measurements made by the writer, and it was the general belief that they constituted the poorest work of the 800 000 m. In fact, they were checked over with the tape because of the large discrepancies from the Mexican measurements.

The writer shares the opinion, expressed in Mr. W. N. Brown's discussion, that the author underestimates the accuracy with which stadia measurements may be made. Moreover, this greater accuracy can be secured without the expense of much labor or time. The office work of preparing an interval table* does not require more than an hour, and with such a table the day's field notes can be reduced in 15 min. The time taken for an interval determination will be a small part of that involved in sending the transit to the instrument maker for adjustment of the wires.

* Johnson and Smith's "Surveying," pp. 76-78.

Mr. Smith.

TABLE 25.—ACCURACY OF STADIA MEASUREMENTS ON BARLOW UNITED STATES–MEXICO BOUNDARY LINE, 1892–94.

(1) No.	(2) Between monuments.	(3) k = 104.60 U. S. stadia, in meters.	(4) Triangulation and steel tape, in meters.	(5) Error of stadia, in meters.	(6) Ratio of error.	(7) k = 104.44 U. S. stadia.	(8) Error of stadia, in meters.	(9) Ratio of error, 1 in:
1	43-44	3 691.74	3 690.14	+ 1.60	2 306	3 686.1	— 4.04	913
2	46-47	4 779.95	4 773.91	+ 6.04	790	4 772.7	— 1.21	3 944
3	48-49	4 445.93	4 443.17	+ 2.76	1 610	4 439.0	— 4.17	1 064
4	62-63	3 340.28	3 334.02	+ 6.26	532	3 335.2	+ 1.18	2 820
5	69-70	3 332.65	3 324.18	+ 8.47	392	3 327.5	+ 3.32	1 000
6	78-79	5 992.19	5 987.53	+ 4.66	1 280	5 983.0	— 4.53	1 320
7	83-84	6 067.67	6 060.42	+ 7.25	830	6 058.4	— 2.02	3 020
*8	84-85	3 983.66	3 977.6	+ 6.06	660	3 977.6	0.0	Infinity.
*9	85-86	4 449.98	4 444.1	+ 5.88	755	4 443.2	— 0.9	4 930
10	86-87	6 332.4	6 325.51	+ 6.89	920	6 322.9	— 2.61	2 420
11	87-88	6 698.31	6 685.18	+ 13.13	509	6 688.2	+ 3.0	2 230
12	91-92	5 698.71	5 693.35	+ 5.36	1 060	5 690.1	— 3.25	1 750
*13	92-93	4 465.2	4 462.58	+ 2.62	1 700	4 458.4	— 4.18	1 060
14	93-94	3 310.4	3 308.87	+ 1.53	2 160	3 305.4	— 3.47	924
15	94-95	3 478.41	3 475.07	+ 3.34	1 040	3 473.2	— 1.90	1 820
16	95-96	3 074.85	3 073.61	+ 1.21	2 540	3 070.2	— 3.41	900
17	96-97	3 627.78	3 623.94	+ 3.84	943	3 622.3	— 1.64	2 200
18	107-108	1 530.27	1 527.33	+ 2.94	510	1 528.0	+ 0.67	2 280
19	119-120	2 475.42	2 471.31	+ 4.11	600	2 471.7	+ 0.39	6 330
	Totals.	80 775.80	80 681.82	+ 94.0	858	80 653.1	— 28.8	2 800

* Stadia measurements referred to by the author in Table 17.

Greater accuracy in the stadia measurements may involve the saving of expensive control.

This is well illustrated by the methods adopted by the Wisconsin Water-Power Survey, a co-operative survey between the State and the U. S. Geological Survey.*

The specifications for this work were made nominally by the U. S. Geological Survey, but the writer was permitted to make several important changes which either added to the value of the work or greatly reduced its cost. One modification in the field work was the substitution of the magnetic needle for the control of all directions, the magnetic declination being frequently determined from Polaris observations. All distances were read by stadia and water levels; shore lines and contours were determined either by level or vertical-angle readings. Only every other turning point was occupied by the transit, a method which increased the speed by about 33 per cent.

Such horizontal control as was needed for the purposes of a preliminary survey was secured by checking on the section lines where

* This work was done in 1906-07.

they crossed the river, a feature which had the additional advantage of showing also the ownership of the riparian rights. Mr. Smith.

The vertical control was secured by running Wye-level lines and establishing permanent bench-marks on or near the river at intervals of about 2 miles, 400 miles of such levels being run at a total expense of \$1 277. The unit cost of these levels varied from \$2.16 per mile in a settled country with good roads to \$4.85 per mile in a region devoid of roads and settlements. The instrument men on this work were junior civil engineering students at the University of Wisconsin.

As little on the subject of the accuracy of transit leveling has appeared in engineering literature, it may be of interest to examine in detail the accuracy of this work, which is shown in Tables 26, 27, and 28. In judging its accuracy, it should be remembered that the average turning-point distance was about 1 500 ft., and that the young men had had very little instrumental experience. The sensibility of the level corresponded to $\frac{1}{10}$ in. = 15″ of arc. The transits had object glasses $1\frac{1}{4}$ in. in diameter and the eye-piece magnified 24 times.

TABLE 26.—ACCURACY OF TRANSIT LEVELING ON BLACK RIVER, WISCONSIN.

No. of check.	Distance, in miles.	Error in Stretch.				Accumulative Error.		Total.
		Plus.	Minus.	Σ Error.	Per mile.	Total.	Per mile.	Miles.
1	2.3	0.09	....	+ 0.09	0.04	+ 0.09	0.04	2.3
2	2.3	0.44	....	+ 0.44	0.20	+ 0.53	0.12	4.6
3	2.4	0.17	0.69	− 0.52	0.20	+ 0.01	0.001	7.0
4	3.6	0.23	0.54	− 0.31	0.09	− 0.30	0.029	10.6
5	4.6	....	0.14	− 0.14	0.03	− 0.44	0.030	15.2
6	5.2	1.50	....	+ 1.50	0.29	+ 1.06	0.050	20.4
7	3.8	....	0.57	− 0.57	0.15	− 0.49	0.020	24.2
8	3.2	1.01	....	+ 1.01	0.31	− 1.50	0.055	27.4
9	3.6	0.72	0.90	− 0.18	0.05	− 1.32	0.042	31.0
10	3.6	0.20	....	+ 1.20	0.33	+ 2.52	0.073	34.6
11	6.3	0.49	0.59	− 0.10	0.02	+ 2.42	0.06	40.9
12	4.8	0.31	0.75	− 0.44	0.09	+ 1.98	0.044	45.7
13	5.2	0.27	0.17	+ 0.10	0.02	+ 2.08	0.041	50.9
14	1.8	....	0.78	− 0.78	0.40	+ 1.30	0.025	52.7
15	5.3	....	0.79	− 0.79	0.14	+ 0.51	0.009	58.0
					Average, 0.15			

Table 26 shows that the largest error per mile on any stretch of the Black River Survey was 0.33 ft., and that the average error per mile was 0.15 ft. These errors, however, compensated to a remarkable extent, so that at the end of 10, 20, 31, 41, 51, and 58 miles the corresponding accumulative errors were only 0.03, 0.05, 0.04, 0.06, 0.04, and 0.009 ft. per mile. The same engineer, Mr. Victor Reineking, next surveyed the Flambeau River with even better results, as will be seen from Table 27.

Mr. Smith.

TABLE 27.—Accuracy of Transit Leveling on Flambeau River.

No. of check.	Distance, in miles.	Error in Stretch.				Accumulative Error.		Total miles.
		Plus.	Minus.	Σ Error.	Per mile.	Total.	Per mile.	
1	3.4	0.27	0.05	+0.22	0.06	+0.22	0.06	3.4
2	2.5	....	0.58	−0.58	0.23	−0.36	0.06	5.9
3	2.8	0.17	....	+0.17	0.07	−0.19	0.02	8.7
4	2.5	....	0.28	−0.28	0.11	−0.47	0.042	11.2
5	1.5	....	0.08	−0.08	0.05	−0.45	0.045	12.7
6	2.2	....	0.10	−0.10	0.045	−0.65	0.043	14.9
7	4.1	....	0.69	−0.69	0.16	−0.134	0.070	19.0
8	4.4	0.62	....	+0.62	0.14	−0.72	0.031	23.4
9	2.7	0.40	....	+0.40	0.14	−0.32	0.012	26.1
10	1.3	0.20	....	+0.20	0.16	−0.12	0.004	27.4
11	1.6	0.22	....	+0.22	0.13	+0.10	0.003	29.0
12	1.9	0.38	....	+0.38	0.20	+0.48	0.015	30.9
13	5.0	....	0.28	−0.28	0.05	+0.20	0.006	35.9
14	2.5	....	0.68	−0.68	0.15	−0.48	0.013	38.4
15	1.2	0.34	....	+0.34	0.22	−0.14	0.004	39.6
16	2.5	0.22	....	+0.22	0.08	+0.08	0.002	42.1
17	2.4	0.02	....	+0.02	0.008	+0.10	0.001	44.5
18	3.1	....	0.27	−0.27	0.087	−0.17	0.004	47.6
19	3.1	0.09	....	+0.09	0.028	−0.08	0.002	50.7
20	4.2	....	0.02	−0.02	0.004	−0.10	0.002	54.9
21	1.0	....	0.06	−0.06	0.06	−0.16	0.006	55.9
22	1.8	....	0.27	−0.27	0.15	−0.43	0.008	57.7
23	3.6	0.51	....	+0.51	0.07	+0.08	0.001	61.3
24	6.6	....	0.03	−0.03	0.005	−0.05	0.001	67.9
25	4.8	0.46	....	+0.46	0.096	+0.51	0.007	72.7
26	1.4	0.11	....	+0.11	0.078	+0.62	0.008	74.1
27	4.0	0.05	....	+0.05	0.012	+0.67	0.008	78.1
28	3.2	0.22	....	+0.22	0.070	+0.89	0.011	81.3
29	3.6	....	0.56	−0.56	0.153	+0.33	0.004	84.9
30	3.0	....	0.04	−0.04	0.012	+0.29	0.003	87.9
31	4.4	....	0.03	−0.03	0.007	+0.26	0.003	92.3

TABLE 28.—Accuracy of Transit Leveling on Wisconsin River.

No. of check.	Distance, in miles.	Error of Stretch.				Accumulative Error.		Total miles.
		Plus.	Minus.	Σ Error.	Per mile.	Total.	Per mile.	
1	18.1	1.14	1.77	−0.63	0.039	−0.63	0.039	18.1
2	14.6	....	2.73	−2.73	0.187	−3.36	0.103	32.7
3	15.0	0.45	....	+0.45	0.03	−2.91	0.061	47.7
4	15.0	....	0.28	−0.28	0.02	−3.19	0.050	62.7
5	27.0	0.45	0.55	−0.10	0.004	−3.29	0.036	89.7
6	8.0	0.04	0.01	+0.03	0.003	−3.26	0.033	97.7
7	16.3	0.16	0.53	−0.37	0.023	−3.63	0.032	114.0
8	6.3	0.10	0.04	+0.06	0.01	−3.57	0.029	120.3
9	25.0	0.86	....	+0.86	0.03	−2.71	0.019	145.3
10	23.0	0.85	0.17	+0.68	0.02	−2.03	0.012	168.3
11	10.7	0.49	0.16	+0.33	0.03	−1.70	0.010	179.0
12	6.0	0.54	....	+0.54	0.09	−1.16	0.006	185.0
13	6.0	0.38	....	+0.38	0.06	−0.78	0.004	191.0
14	9.0	0.34	0.13	+0.21	0.02	−0.57	0.003	200.0

Mr. Smith.

TABLE 29.—ACCURACY OF TRANSIT LEVELING ON PESHTIGO RIVER.

No. of check.	Distance, in miles.	Error of Stretch.				Accumulative Error.		Total miles.
		Plus.	Minus.	Σ Error.	Per mile.	Total.	Per mile.	
1	2.1	0.16	....	+0.16	0.08	+0.16	0.08	2.1
2	1.9	....	0.17	+0.01	0.09	+0.01	0.002	4.0
3	3.6	0.72	....	+0.72	0.19	+0.71	0.09	7.6
4	1.5	0.13	....	+0.13	0.09	+0.84	0.09	9.1
5	4.7	0.32	....	+0.32	0.07	+1.16	0.08	13.8
6	2.5	0.13	....	+0.13	0.05	+1.29	0.078	16.3
7	5.0	0.02	....	+0.02	0.004	+1.31	0.062	21.3
8	5.0	0.16	....	+0.16	0.032	+1.47	0.056	26.3
9	7.6	0.17	....	+0.17	0.022	+1.64	0.049	33.9
10	4.0	0.0	0.94	−0.94	0.23	+0.70	0.018	37.9
11	5.1	....	0.46	−0.46	0.09	+0.24	0.005	43.0
12	2.4	....	0.04	−0.04	0.017	+0.20	0.004	45.4
13	3.5	0.81	....	+0.81	0.23	+1.01	0.02	48.9
14	1.3	....	0.03	−0.03	0.02	+0.98	0.02	50.2

The largest error was 0.22 ft. per mile, the average being 0.09 ft. per mile. Of the 31 checks, 14 show minus and 17 plus errors. This compensation, as shown in Table 27, is truly surprising. The total accumulative error at the end of the 92.3 miles was only 0.26 ft. or 0.027 ft. per mile.

Table 28 gives the results of transit leveling on the Wisconsin River Survey. Because of a misunderstanding of directions, the leveling on the first 32.7 miles of river was done by reading vertical angles, with the result that the largest errors are found in this stretch. In the remaining 168 miles of levels, the average error per mile is 0.04 ft., and the total accumulative error on the 200 miles is 0.57 ft.

The 82 miles of transit levels run on the Peshtigo River were, for the most part, in rough, rocky, and heavily timbered country. The largest error per mile in any stretch was 0.23 ft., the smallest was 0.02 ft., with an average of 0.09 ft. The largest accumulated error was 1.64 ft., at a point 33 miles from the beginning. The accumulative error per mile gradually decreased from 0.09 ft. near the beginning to only 0.02 ft. at the end of the 82 miles.

It is perfectly manifest, from this record of 400 miles of transit levels, that for general preliminary surveys transit levels are amply sufficient without Wye-level control. Such omission would have reduced the cost of the survey by an average of $3.24 per mile.

Very careful records of the most important elements of cost data were kept, and are given in Tables 30 and 31. These elements, together with the costs of supervision, are assembled in Table 31. The greater cost of the Flambeau River work was due to working in the winter on the ice. The winter happened to be one of unusual severity, with very low temperatures and many deep snows. This resulted in the loss

Mr. Smith.

TABLE 30.—SUMMARY OF TOTAL AND UNIT COSTS OF CO-

River surveyed.	Spirit Levels.								Topography.					
	Total miles.	Average per work day, in miles.	Lost time.		Miscellaneous.		Gross cost.		Total miles.	Average per work day.	Lost time.		Miscellaneous.	
			Days.	$	Days.	$	Total.	Per mile.			Days.	$	Days.	$
Wisconsin	1 873	4.9	4	35	4	35	408	2.16	197.0	2.5	16	142	15	135
Eau Claire	.....	.....	.....	.....	.....	.....	.....	....	25.8	3.2	10	88	3	18
Peshtigo	40	5.0	7	59	5	38	194	4.87	81.7	3.3	1	12	18	182
Black	69	3.9	4	36	1	9	207	3.00	63.0	3.0	6	50	1	8
Flambeau	102	3.9	9	99	5	55	467	4.58	119.5	3.0	15	165	9	99
Totals and averages.	3 983	4.5	24	229	15	137	1 271	3.19	487.0	2.86	48	457	41	442

NOTE :—"Lost time" includes Sundays, holidays, and stormy weather. Miscellaneous

of about 30 work days, which increased the cost by $3.50 per mile of survey.

The surveys were mapped on a scale of 1 000 ft. to 1 in., and published by the U. S. Geological Survey on a scale of 2 000 ft. to 1 in. It will be noted that the total cost of 487 miles of river surveys, including both field and office expenses, was $4 798.30, or only $9.85 per mile for both profile and plan with land sections. This work furnishes a good example of the adaptability of the transit and stadia method.

TABLE 31.—TOTAL AND UNIT COST OF WISCONSIN CO-OPERATIVE RIVER SURVEY, 1905-06.

L. S. SMITH, Hydrographer in Charge.

River surveyed.	Spirit Leveling.			Topography.			Map and Profiles.			Cost of reconnaissance and supervision, per mile.	Total cost per mile of river.	Total cost.
	Total miles.	Total cost.	Cost per mile.	Total miles.	Total cost.	Cost per mile.	Total miles mapped.	Total cost.	Cost per mile.			
Wisconsin	187.3	$403	$2.16	197.0	$981	$4.97	197.0	$171	$0.89	$0.73	$8.61	$1 696.00
Eau Claire	0.0	0	0.00	25.8	177	6.87	25.8	11	0.44	1.00	8.29	213.80
Peshtigo	40.0	194	4.85	81.7	521	6.36	81.7	40	0.50	0.70	9.98	815.20
Black	69.3	213	3.10	63.0	213	3.38	63.0	57	0.90	1.10	8.76	552.30
Flambeau	102.0	467	4.58	119.5	796	6.69	119.5	106	0.90	1.25	12.71	1 519.00
Total	398.6	$1 277	$3.24	487.0	$2 688	$5.52	487.0	$385	$0.79	$0.90	$9.85	$4 798.30

Method—Transit and stadia, with Wye-level control.
Scale—1 in. = 1 000 ft.
Contour interval = 10 ft. on land and 1 ft. on water.
Section lines located frequently.

OPERATIVE WATER-POWER SURVEY IN WISCONSIN, 1905-06. Mr. Smith.

Gross cost.		Mapping.						Time not Spent in Field Work.							
		Net cost.		Gross cost.		Total per mile.		Lost time.		Miscellaneous.		Σ Time.		Σ Cost of field and office work.	
Total.	Per mile.	Map.	Profile.	Map.	Profile.	Map.	Profile.	Days.	$	Days.	$	Days.	$		
981	4.97	120	11	160	16	0.81	0.08	22	182	25	210	47	392	$1 560	Dugan.
177	6.87	9	2	9	2	0.86	0.06	10	88	3	18	13	106	188	"
521	6.38	32	3	37	3	0.46	0.04	10	76	21	221	31	297	755	"
248	3.94	....	....	51	6	0.80	0.10	10	90	2	18	12	108	511	Reineking.
768	6.43	....	....	94	12	0.80	0.10	24	264	14	54	38	318	1 341	"
2 695	5.53	....	....	351	39	0.72	0.08	76	700	65	521	141	1 121	$4 355	

time includes traveling and all causes not included in "lost time."

W. W. FOLLETT, M. AM. SOC. C. E. (by letter).—Owing to illness and absence from his office, where all his records are kept, the writer can only review the discussion of this paper briefly. The interest shown by the members in the subject has been gratifying. Mr. Follett.

Mr. Landreth brings out quite plainly the relation between the number of shots per square mile and the cost. His figures, however, as would be expected, show that the cost does not increase in direct ratio to the number of shots. With 15 times as many shots as on the work described by the writer, his cost was only 4½ times as much per square mile. Of course, conditions as to brush, etc., may have affected the relative cost of the two pieces of work.

After further study, the writer is inclined to agree with Mr. Brown that he set the maximum errors too high, and will modify his statement on that subject to read:

"In any work where a variation of ½ m. to 1 m. in the relative location of points near together, or 3 or 4 m. in that of those which are material distances apart, can be tolerated, the stadia offers a most rapid and handy method of work."

Beyond this, he cannot go. The Cincinnati surveys must have been most thoroughly controlled by triangulation. Even then, the writer does not see how a map could be made, which would scale so closely under all conditions of weather. He has found that map paper swells when the air is moist and that the swelling crosswise of the sheet may be twice that lengthwise, so that the map is distorted, and measurements of long distances cannot be checked to the limit mentioned of "$\frac{1}{80}$ of an inch".

The rod used permitted of accurate readings, under good atmospheric conditions. The work was not done in a slipshod manner,

Mr. Follett.

as might be inferred from Mr. Brown's remarks. Certain standards, which were considered proper for obtaining data for a map of 1 in 5 000, were adopted and enforced.

The writer regrets that he did not note the table on page 235 of General Barlow's report, and reproduced by Mr. Smith, as Table 25. He took the distances between monuments from the official report, signed by all six commissioners—three on each side—supposing them to be correct. It is gratifying to know that his chaining was so good.

Two of the Brandis transits used on this Barlow work fell to the writer, and he used them on an extensive survey of the lower Rio Grande in 1897-98. They were a source of constant worry, as the stadia wires would vary slightly in their intercept and in neither was the interval 1 to 100, nor was the middle wire half way between the other two. Frequent interval determinations were necessary.

The use of the needle, as suggested by Mr. Smith, does not appeal to the writer, except for very rough preliminary surveys. He does not believe that any man can read a needle closer than to 5′ of arc, and his error may be 10′, and an error will be introduced which is many times that which arises from the neglect of $f + c$. The diurnal variations will also affect the needle readings. This may do for very rough work, but not where a fair degree of accuracy is desired.

It was not the writer's intention to intimate that he sent instruments to the makers for adjustments other than that of the stadia wires. In fact, he has never sent an instrument to the maker for any adjustment. If the stadia wires did not subtend 1 in 100, he used a reduction table. He merely suggested the expediency of having wires reset.

The point in the paper most criticized was the ignoring of $f + c$ and the absence of interval determinations. The writer should have stated that the intervals of the transits used on the stadia work were tested at the beginning and at the close of the survey, and that one, with which the instrumentman did not get as good closures as the other men, was tested two or three times during the progress of the work.

Assuming that $f + c$ is to be ignored in the field work, the writer would enquire how a man is to proceed to make a reduction table when, the hubs having been accurately set by steel tape 50 m. apart and the transit set up $f + c$ (which was, for the transits used, if the writer's memory serves him, about 0.3 m.) back of the first hub, the instrumentman states that he reads exactly 50 cm. on the 50-m. hub, exactly 1.00 m. on the 100-m. hub, etc., and when the other men present, including the writer, check these readings? Three of the five transits used on the survey gave this result. It was known that the intervals

of the other two were not 1 in 100, but, as they were not to be used on stadia work, their intervals were not determined. Mr. Follett.

As for the ignoring of $f + c$, the results given in the paper, in the writer's opinion, are sufficient to show its expediency.

The writer agrees with Mr. Brown that the quoting of cost data on any work, stadia or other, is a dangerous proceeding unless all the conditions are fully stated, and that even then it may be misleading. It was the intention to compare the cost of this with similar work. He cannot, however, imagine any case where he would feel justified in spending from \$1 000 to \$2 000 per sq. mile, or even the first amount on a stadia survey. If property is so valuable that such an expense in making a survey of it is justified, then some method more accurate than stadia should be used.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 1302

REINFORCED CONCRETE RESERVOIR AND COAGULATION PLANT AT ST. LOUIS, MO.*

By Edward Flad, M. Am. Soc. C. E.

With Discussion by Messrs. J. K. Finch, Alexander Potter, Charles B. Buerger, A. W. Buel, Edward Wegmann, and Edward Flad.

Before describing the design and construction of the reservoir and coagulation plant which are the subjects of this paper, a short explanation of the water-works system to which they belong may not be amiss.

The City of St. Louis has guarded zealously the right to supply water to its citizens, but it has no power to prevent the owners of riparian rights from taking water from the river and leading it to industries on the river front, if no streets, alleys, or public places are crossed or entered on by the pipe lines. Thus, there has developed, in connection with the Anheuser-Busch Brewery, in that city, a water-works system with a capacity of more than 6 000 000 gal. per day, or sufficient to supply the needs of a city having a population of 80 000 people.

The city is perhaps justly proud of the improvement made a few years ago in the quality of its water supply, which improvement was accomplished by the simple process of dumping into the water suitable chemicals which hasten sedimentation. This just pride should be

* Presented at the meeting of February 4th, 1914.

tempered, however, by the knowledge that, as early as 1901, several years before the city fathers could be prevailed on to adopt modern methods, the Anheuser-Busch Brewery had pointed out the way, by taking its water supply from the Mississippi River and clarifying it by the use of a chemical coagulant and rapid filtration—a further refinement of the method adopted later by the city.

The water-works of the Anheuser-Busch Brewery are on the river front, south of Dorcas Street. The water is taken from the Mississippi through two 20-in. cast-iron intake pipes, and is siphoned into one of two intake wells, from which it is pumped into settling tanks. It flows by gravity through the settling tanks, thence through the filters, and is then pumped into the distribution system.

The low-service pumps are in a brick pit, 30 ft. in diameter and 40 ft. deep. There are three centrifugal pumps having a combined capacity of 13 000 000 gal. per day, and one triplex, direct-acting pump having a capacity of 2 000 000 gal. per day.

There are two steel settling tanks, 75 ft. in diameter and 28 ft. high, one circular concrete reservoir approximately 150 ft. in diameter and 30 ft. deep, and one rectangular covered reservoir having a capacity of about 1 000 000 gal.

The filter plant comprises six Jewell filters, circular in plan, each 16 ft. in diameter, and three Reisert filters, recently completed, which are rectangular in plan, each being approximately 34 by 15 ft.

Chemical Treatment.—The water is settled by adding sulphate of aluminum (alum) and lime. A special three-story reinforced concrete building is provided for storing and preparing the chemicals.

The hopper for storing the lime is 36 by 9 by 14 ft. high, and has a capacity of 90 tons. It is placed in a pit so that it can be filled directly by shoveling from the cars. An electric elevator conveys to the third floor the hand-cars containing the lime or alum.

The alum is dissolved in a concrete tank, and is fed to the water by gravity. This alum tank has three rectangular divisions, each 10 by 7 by 5 ft. deep. Each division is charged with from 500 to 2 000 lb. of alum which is dissolved in water. It requires from 2 to 5 hours to dissolve one charge. The lime is slaked in iron tanks on the third floor. These tanks are rectangular, 12 by 12 by 7 ft. deep, with sloping bottoms. A false perforated bottom is provided at a depth of 30 in., on which the lime is placed and partly submerged in 1 ft. of water.

After slaking, which requires about 1 hour, the attendant stirs the mixture, which passes readily through the perforated bottom and into the lime tanks below. This milk of lime is stored in three vertical cylindrical iron tanks, 12 ft. in diameter and 18 ft. high, with conical bottoms. One charge of lime consists of from 2 000 to 6 000 lb., and the tank holds about 10 000 gal. of water, giving a 2½ to 7% solution. In these tanks the milk of lime is kept agitated by compressed air admitted at the bottom through a small perforated pipe.

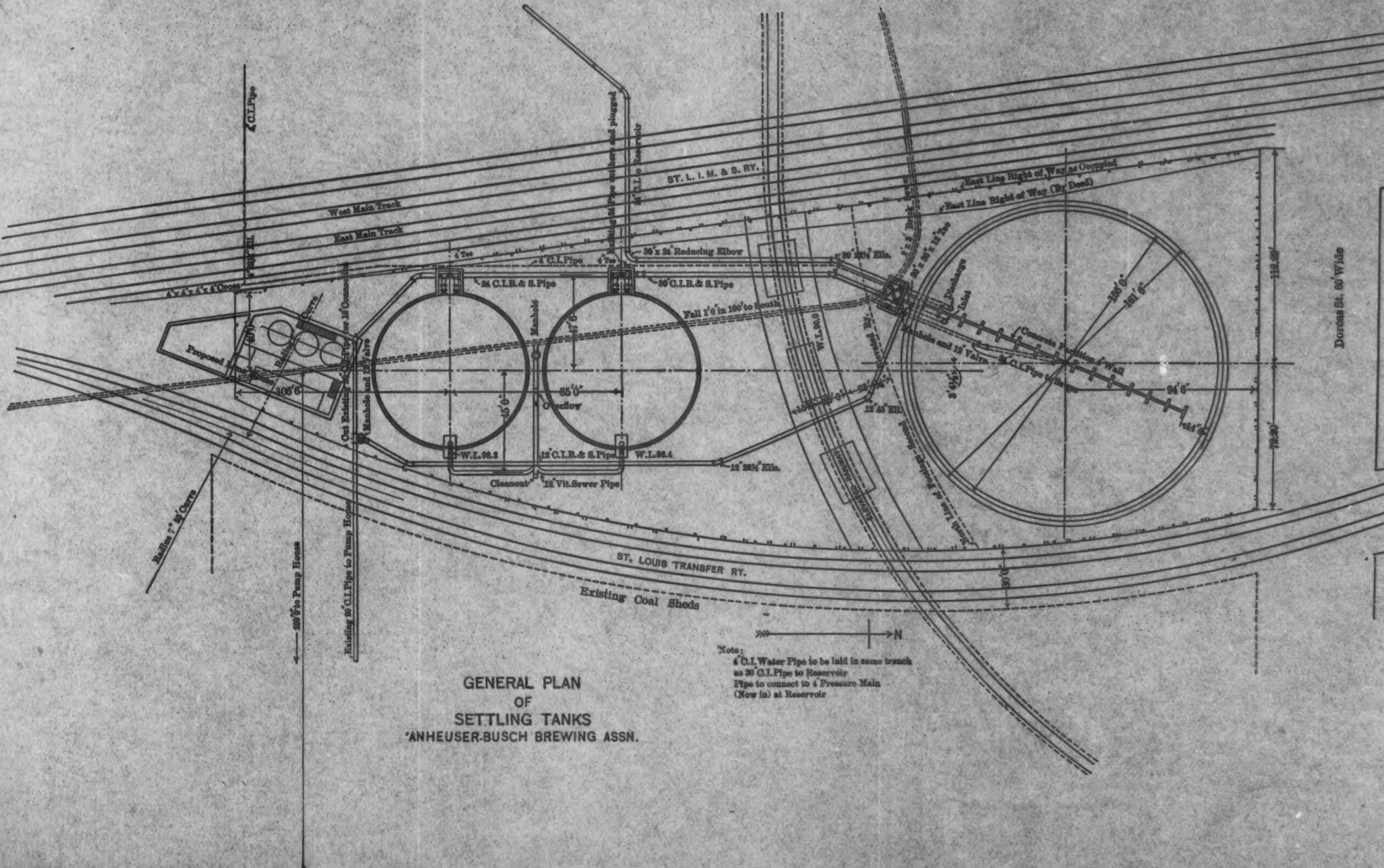
The flow of both the alum and lime solutions is regulated by standardized orifices operating under fixed heads. The milk of lime is pumped into the supply pipe or settling tanks by a centrifugal pump. As a general rule, the lime is added to the water as it enters the first settling tank, and the alum as the water enters the second settling tank.

Consumption.—The maximum consumption of water during the summer is about 6 000 000 gal. per day, for which at times, 1 600 lb. of alum and 5 600 lb. of lime are required, or about 2 grains of alum and 7 grains of lime per gallon of water.

The water is used for boiler purposes, for condensing and cooling in connection with the ice machine, as well as for washing barrels, bottles, etc. It is not used in brewing beer, partly because of the natural prejudice against water taken from the river below the outlet of the large city sewers.

Reinforced Concrete Reservoir.—Reverting now to the reinforced concrete reservoir recently completed: After due consideration of the various possibilities as to the shape and location of the reservoir, a circular shape was decided on, and the diameter was made as large as the available space permitted. The elevation of the top was fixed by that of the water in the old settling tanks, which operate in series with the new reservoir; and the bottom was placed sufficiently low to pass below the fill of cinders and rubbish, and rest on the river silt.

Estimates were made of the comparative cost of a steel tank and a reinforced concrete reservoir. Exclusive of the foundations, pipes, gate-house, and accessories common to both designs, the steel tank was estimated to cost $32 000, and the reinforced concrete reservoir, $30 800. The reinforced concrete reservoir was supposed to have some advantage, being a more permanent form of construction and not requiring painting, and perhaps a desire to follow the latest fashion



GENERAL PLAN
OF
SETTLING TANKS
ANHEUSER-BUSCH BREWING ASSN.

Note:
4" C.I. Water Pipe to be laid in same trench
as 30" C.I.Pipe to Reservoir
Pipe to connect to 4" Pressure Main
(Now in) at Reservoir

had a minor influence; at all events, it was decided to use reinforced concrete.

The reservoir has vertical sides and is 153 ft. 6 in. in diameter at the top and 35 ft. deep at the center. The side-wall extends 25 ft. 6 in. above the ground. The capacity is approximately 4 250 000 gal. There is a central partition, consisting of a 4-in. reinforced concrete wall with buttresses, which starts at one side of the reservoir and passes diametrically across to within 14 ft. of the other side. The object of this partition is to make the entering water circulate around the reservoir before reaching the outlet. The diameter of the intake and outlet pipes is 30 in., and that of the waste pipe 24 in. The outlet pipe has a float and a hinged joint, so that water is always taken from near the surface. The valves controlling the flow are in a gate-chamber outside the reservoir.

Foundation.—The foundation is a 12-in. layer of concrete resting directly on the river silt and reinforced, in two directions at right angles, with 1-in. square bars, 2 ft. from center to center, making ½% reinforcement each way.

On top of this foundation rests the bottom of the reservoir, which is 6 in. thick and reinforced in a manner similar to the foundation, except that the bars are ½-in. square and 6 in. from center to center.

The top of the foundation was coated with a thin layer of coal-tar, the object being to provide for expansion and contraction of the bottom independent of the foundation.

Side-Walls.—The thickness of the concrete side-walls is 7 ft. 6 in. at the base, 2 ft. 5 in. at the ground line, and 12 in. at the top. The pressure of the water is carried by hoop tension, the side-walls being reinforced circumferentially with corrugated round bars, under the assumption that the concrete carries no tension. There are three lines of 1¼-in. round bars at the bottom and two lines of ¾-in. round bars at the top, the sections and spacing being varied from bottom to top to correspond with the pressure. It was intended to allow a stress of 15 000 lb. per sq. in. on the bars. Owing to an error in the calculation, which, happily, was on the safe side, an excess of steel was used, making the stress per square inch somewhat less than that originally contemplated.

The thickness of the concrete wall was fixed at each point so that the concrete would not be stressed more than 290 lb. per sq. in., under

the assumption that no vertical cracks would develop under this tension. Assuming the modulus of elasticity of steel at 30 000 000 and concrete at 3 000 000, the actual maximum stress, if no vertical cracks develop, will be 2 900 lb. per sq. in. of steel and 290 lb. per sq. in. of concrete.

The following formulas were used in determining the dimensions and spacing of the steel bars and the thickness of the concrete.

A = Area of bars, in square inches per foot of height of wall;
D = Vertical distance, in inches, between two layers of bars at the point selected;
T = Thickness of concrete wall, in inches, at the point selected;
p = Water pressure, in pounds per square foot, at the point selected;
r = Radius of reservoir, in feet;
s = Stress in steel, in pounds per square inch, allowed, under the assumption that the concrete carries no tension;
c = Stress in concrete, in pounds per square inch, allowed;
a = Area, in square inches, of steel in each layer.

The area of the bars required per foot of height of wall is

$$A = \frac{p\,r}{s}$$

The vertical distance between the layers of bars is

$$D = \frac{12\,a}{A}$$

The thickness of the concrete wall at any point is

$$T = \frac{p\,r - 9\,A\,c}{12\,c}.$$

The reinforcing bars were held in position by angle-iron frames, the sides of the angles being punched accurately with small holes for the insertion of the wires which tied the bars to the frame.

Splices.—The circumferential bars were in lengths of from 50 to 55 ft., and were spliced by lapping forty diameters and attaching two Crosby clips at each lap. The strength of this joint was tested by making up sample joints and pulling the bars in a testing machine. Two tests were made on each diameter of bar used, one with bars and clips without any mortar, and the other with the same joint embedded in cement mortar of the same mixture as that used for the walls of the reservoir.

PLATE XIX.
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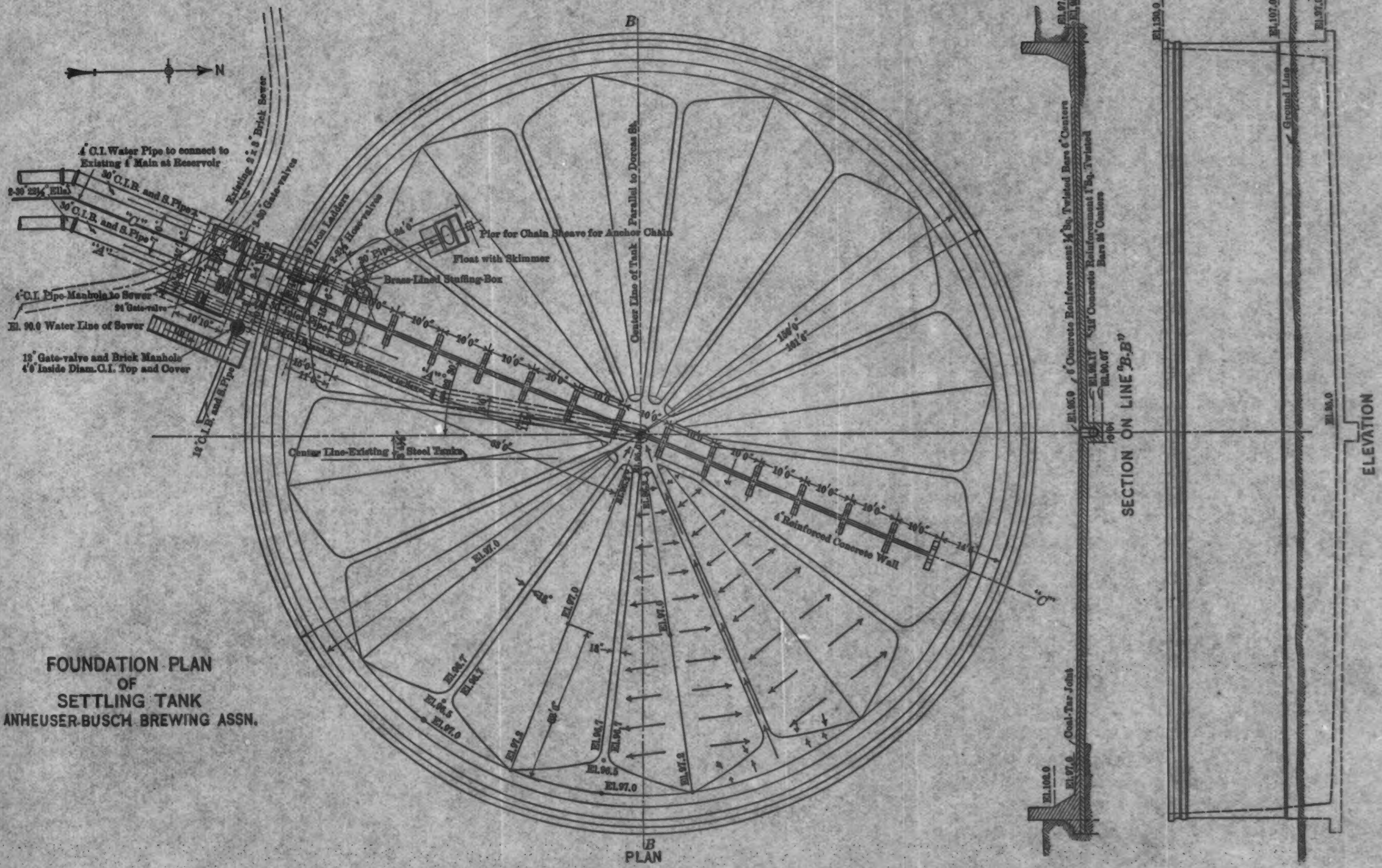

FOUNDATION PLAN
OF
SETTLING TANK
ANHEUSER-BUSCH BREWING ASSN.

FIG. 1.—WOODEN FORMS FOR SIDE-WALLS OF RESERVOIR, STAR-SHAPED BOTTOM, AND VALVES ON INLET AND OUTLET PIPES.

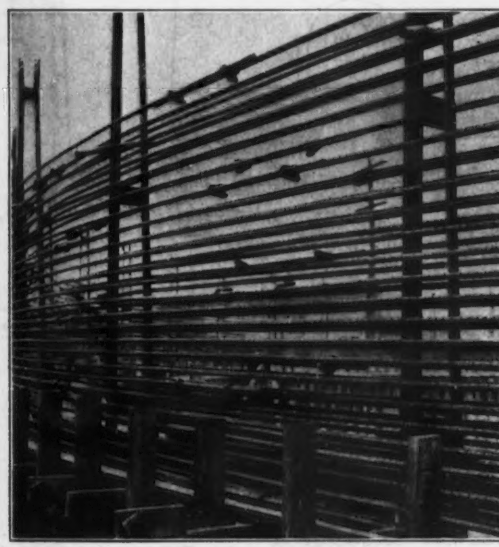

FIG. 2.—BARS FOR SIDE-WALLS OF RESERVOIR, WITH ANGLE-IRON SUPPORTS FOR BARS, ETC.

PLATE XX.
TRANS. AM. SOC. CIV. ENGRS.
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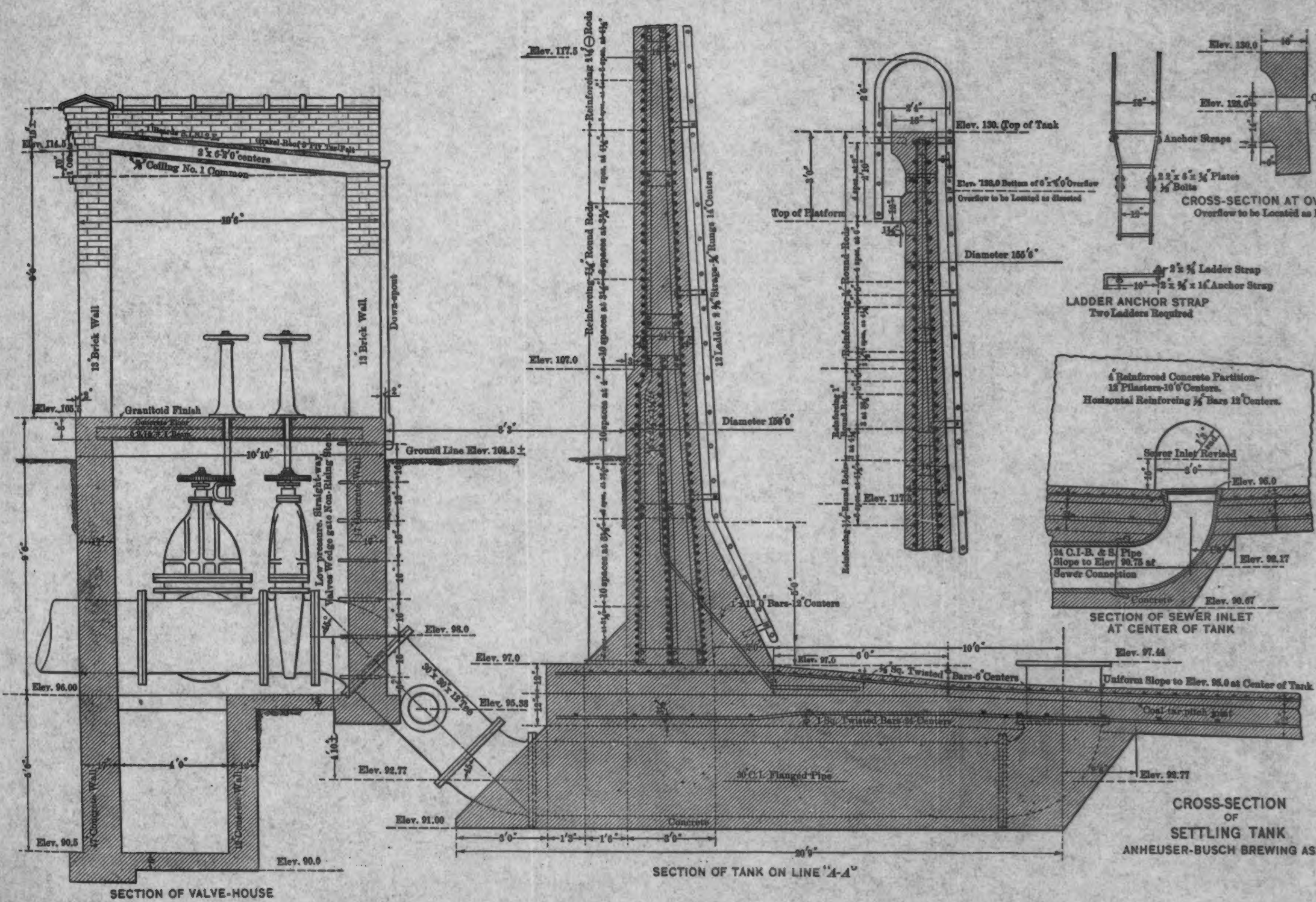
FLAD ON
REINFORCED CONCRETE RESERVOIR.



CROSS-SECTION
OF
SETTLING TANK
ANHEUSER-BUSCH BREWING ASSN.

The tests showed that the presence of the mortar did not add materially to the strength of the joint, as it failed either by slippage or by breaking off the clip. The first sign of slippage occurred at a stress of from 18 000 to 35 000 lb., and the ultimate strength of the joint varied from 25 000 to 75 000 lb.

Table 1 gives the results of the tests.

TABLE 1.—TESTS OF BARS.

Kind of bar.	Began slipping.	Broke at.	Remarks.
1¼-in. round bar, plain	....	75 600 lb.	1-in. slip at 18 000 lb.
" " " with mortar.	....	40 500 "	Mortar broke at 29 000 lb.
1⅛-in. round bar, plain	37 400 lb.	....	
" " " with mortar.	32 200 "	32 200 "	
1-in. round bar, plain	35 000 "	....	
" " " with mortar ...	25 600 "	25 600 "	
¾-in. round bar, plain	27 300 "	30 000 "	
" " " with mortar...	26 400 "	26 400 "	

At the junction of the side-wall with the bottom, knee-bars are provided to reinforce this connection. These bars are 1-in. square and 12 in. from center to center.

Mixture.—The concrete in the walls and bottom is a mixture of 1 part cement, 1½ parts sand, and 3 parts limestone screenings run through a ¾-in. mesh screen, with the addition of 10½ lb. of Shamrock water-proofing to each barrel of cement. The composition of this water-proofing is approximately as follows:

Silica..................................	60.0	per cent.
Alumina................................	15.0	" "
Lime.....................................	6.5	" "
Oxide of iron........................	1.5	" "
Combined water...................	10.0	" "
Gelatinous material...............	7.0	" "

This material cost 73½ cents per bbl. of cement.

Forms.—The forms for the side-walls were built up of ⅞-in. tongued and grooved flooring, with 2 by 6-in. vertical studs about 2 ft. from center to center. The studs on the opposite sides were fastened together with iron wire passing through the forms. These wires were approximately 2 ft. apart horizontally and 4 ft. vertically.

The concrete was placed in the forms with a tower and dump-cars running on a circular track.

Arrangements for Washing Out.—The bottom of the tank slopes toward the center from all sides, the 24-in. waste outlet being at the center. The slope toward the center could not readily be made more than 2 in 75, being limited by the depth of the sewer which was already built. In order to facilitate the removal of sediment, therefore, the bottom was laid out in a series of star-shaped mounds, as indicated on Plate XIX, each mound draining into a shallow gutter. Hose connections furnish water under pressure for cleaning out.

Bond in Concrete.—Special precaution was taken to obtain a good bond between the successive layers of concrete. The old surfaces were scrubbed with brushes and a stream of water. In addition, 6-in. strips of corrugated plates, about $\frac{1}{16}$ in. thick, were placed vertically in the joints. In spite of these precautions, when the reservoir was filled, small leaks developed along most of the joints, and efflorescence was quite extensive. These leaks appear to be closing up gradually, and it is probable that in the course of time they will disappear entirely. The intention, however, is to empty the reservoir and treat the joints with some water-proofing compound.

After completion the outside of the wall was rubbed with carborundum blocks and brushed with cement mortar.

The reservoir was designed by the writer's company, and was built under contract by the Fruin-Colnon Contracting Company. The cost, including pipes and accessories, was approximately \$52 000. The unit prices and total cost of the various classes of work were as follows:

Excavation, 7 801 cu. yd. at \$0.80	\$6 240.80
Plain concrete, Class A, 58.2 cu. yd. at \$4.00	232.80
" " " B, 281.3 " " " 5.00	1 406.50
Reinforced concrete, 12-in. base, 759 cu. yd. at \$5.00	3 795.00
Reinforced concrete, 6-in. bottom, 473.1 cu. yd. at \$8.00	3 784.80
Reinforced concrete side-walls, 1 178.3 cu. yd. at \$11.00	12 961.30
Reinforced concrete partition, 120 cu. yd. at \$20.00	2 400.00
Reinforcing bars, 582 668 lb. at 1¾ cents	10 196.69
Clips for reinforcing bars	650.00
Ladders and angle-iron supports	852,16
Cast-iron pipes and valves	7 283.00
Gate-house	1 200.00
Miscellaneous	844.59
Total cost	**\$51 847.64**

DISCUSSION

J. K. FINCH, JUN. AM. SOC. C. E. (by letter).—In some cases reinforced concrete has been substituted for steel, in the construction of tanks and stand-pipes, because it was believed it would result in economy, both in first cost and maintenance, and because such structures are generally on high ground, where they can be seen from afar, and it was hoped that a more satisfactory architectural effect could be secured by its use. The first of these expectations has generally been realized, but not the latter, not on account of the material, which is admirably suitable for architectural treatment, but because the failure to obtain a truly water-tight construction causes the marring of the surface of the structure by the formation of moist spots, due to "sweating," and patches of efflorescence. It has been pretty well established that the chief difficulties are: Mr. Finch.

1. Poor concrete;
2. Vertical cracks in concrete due to excessive stretching of steel under hoop tension;
3. Horizontal cracks between days' work, due to difficulty of joining old and new concrete, and the fact that concrete shrinks in setting and the successive layers are held apart by the vertical steel necessary for supporting the hooping rods; and
4. The formation of a horizontal crack, about 5 ft. above the base, in all cases where the bars in the floor or base have been turned up into the side-walls.

The remedy for the first is good materials and workmanship. A solution of the second difficulty has been proposed by H. B. Andrews, M. Am. Soc. C. E., in a paper entitled "A New Theory for the Design of Reinforced Concrete Reservoirs,"* presented before the Boston Society of Civil Engineers in 1910. This is the method followed by Mr. Flad in the design of the St. Louis tank. The writer desires to call attention to the fact that Mr. Flad, however, has used a working tensile strength for the 1 : 1½ : 3 limestone concrete of 290 lb. per sq. in., which appears to be excessive. It must be borne in mind, of course, that economy demands the use of a low factor of safety, say 1½, for the concrete, which is ample, as the ultimate safety of the structure depends on the steel; but this figure, 290 lb., has been taken apparently from Mr. Andrews' paper, in which he proposed this stress for a 1 : 1 : 2 concrete, not a 1 : 1½ : 3 mixture.

It is unfortunate that there are so few tests of concrete in tension on which to base a satisfactory working stress. The tests that have been made show a considerable range in values, due to the effect of different aggregates, methods of mixing, etc., and, generally, only un-

* *Journal*, Assoc. Eng. Soc., Vol. XLVI, 1911, p. 391.

Mr. Finch. reinforced specimens have been tested. M. Considère made a number of experiments on reinforced concrete in tension, and concluded that the action of the reinforcing steel was to distribute the stress throughout the concrete and enable the latter to take a tensile loading considerably higher than the plain unreinforced material. It was shown later, however, that this was not proved by the tests, as M. Considère's method of computing the stress in the concrete and steel was incorrect. In fact, it was shown that a reinforced specimen developed cracks, and the concrete practically failed at about the same load as an unreinforced specimen. A few tests have been made at Columbia University on 6 by 6-in., 1:3:5 briquettes reinforced with 0.68 to 4.26% of steel, which showed a failure stress in the concrete, based on the break in the stress-strain curve of 205 lb. per sq. in., which is about the same as would be expected from an unreinforced specimen.

Mr. Andrews obtained his figure, 290 lb. for 1:1:2 concrete, from a few tests he had made, which gave an average of 281 lb. per sq. in., by increasing his test value by 25 and 10%, which gave him 386 lb., which he considered as the ultimate strength for a large reinforced specimen, and that 290 lb. was sufficiently lower than this to be used in the design. The increase of 25% was to allow for "the usual increase in strength of large size over small specimens," his test pieces being 4 by 4 in. in cross-section; the further increase of 10% was to allow for the effect of reinforcing steel, and was probably based on M. Considère's deductions. Neither of these increases seems to be justified by any tests yet made, but Mr. Andrews' tests did give rather low values. The following stresses are estimated to be fair values for the quality of concrete used in tank work:

Mix.	Age, in days.	Ultimate strength, in pounds per square inch.
1:1 :2	30	300-400
	90	400-500
1:1½:3	30	250-350
	90	350-450
1:2 :4	30	200-300
	90	300-400

The working stress should depend on the length of time to elapse before the tank will be filled, but 290 lb. per sq. in. appears to be high for a 1:1½:3 mixture.

There seems to be no remedy for the difficulty due to field joints, except pouring in one operation, which should be done whenever possible. Proper care in cleaning the surface of the old concrete, and the use of copper strips or dams, will doubtless do much to remedy this trouble.

The formation of a horizontal crack, in cases where the bottom reinforcement has been run up into the sides, is due to the fact that the lower part of the side-walls of the tank is prevented from stretching

Mr. Finch.

under the hoop tension, due to the restraint of the bottom reinforcement. Indeed, the writer is of the opinion that in the lower part of Mr. Flad's design the wall acts as a cantilever in carrying the water pressure, and little or no hoop tension exists. He cannot see why this method of carrying up the bottom reinforcement has been followed in most cases. There is apparently no reason for not designing a tank simply as a vertical section of pipe resting on, but independent of, the base, as shown in Figs. 3 and 4. The pressure on the joint between the sides and base should be investigated, of course, for the effect of wind and also for possible upward water pressure, but, in the majority of cases, this joint will always show an ample positive pressure.

The writer has used the form shown in Fig. 3 in preparing a design for his students in reinforced concrete, and has since learned that Mr. P. D. Johnson, Hydraulic Engineer of the Ontario Power Company, Niagara Falls, has made tests of the type of joint shown in Fig. 4, and has actually constructed the No. 2 Surge Tank of the Ontario Power Company on this principle. This tank is 70½ ft. high and 79 ft.

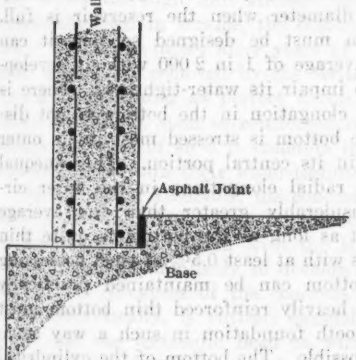


FIG. 3.

FIG. 4.

in diameter, and no cracks have developed. The writer understands that a thin coating of clay was placed on the base under the walls before the wall concrete was deposited.

Mr. Potter.

ALEXANDER POTTER, ASSOC. M. AM. SOC. C. E. (by letter).—As far as the writer knows, this is the largest reinforced concrete reservoir in America of a type in which the hydrostatic pressure is resisted by ring tension; therefore it is of more than usual interest to the Profession.

In reference to reservoirs of this type, especially when approaching such a size, the writer has always felt that the importance of the con-

Mr. Potter. nection of the cylindrical shell to the bottom is not always fully appreciated by the designer, especially as on it, more than, perhaps, on any other element of design, depends the success of the completed structure. The importance of this connection is evident from the fact that in such a reservoir, 150 ft. in diameter, the shell when filled with water expands so as to increase its diameter approximately 0.92 in.

Thus far, no attempts have been made to standardize this feature of the design. Two distinct methods, however, have been evolved in practice to take care of the expansion of the shell. In one, the bottom and shell are constructed as a monolith, and in the other the shell is entirely separated from the bottom by a water-tight expansion joint. The first of these methods is in common use, with more or less success, and is the one which seems to have been used successfully in constructing this settling tank.

Such a rigid connection, to be successful, calls for heavy steel reinforcement throughout the entire bottom, which, for economy, must be of as thin a section as possible so that, by stretching, it can readily adapt itself to the increase in diameter when the reservoir is full. This reinforced concrete bottom must be designed so that it can stretch in all directions on an average of 1 in 2 000 without developing cracks of sufficient extent to impair its water-tightness. There is every reason to believe that the elongation in the bottom is not distributed uniformly, and that the bottom is stressed most in its outer portion and very little, if any, in its central portion. This unequal distribution would tend to give radial elongations in the outer circumference of the bottom considerably greater than the average value. It appears, however, that as long as the bottom is quite thin and is reinforced in all directions with at least 0.5% of steel possessing a high elastic limit, such a bottom can be maintained practically water-tight. Ordinarily, such a heavily reinforced thin bottom must be supported on a rigid and smooth foundation in such a way as to permit of as free expansion as possible. The bottom of the cylindrical shell must also be heavily reinforced to resist the stresses due to the bending moments set up in it by the restraining action of the bottom.

A rigid connection, such as used in the St. Louis reservoir, to be successful, is wasteful of structural material. It would appear to be more economical to use some type of water-tight expansion joint between the shell and the bottom, so as to permit the shell to expand irrespective of the bottom. This latter form of construction has already been used to some extent in circular tanks of relatively small diameter. A discussion on this point is invited, as the use of such expansion joints in tanks of large size, if proven successful, will tend to a more economic form of construction.

Another point on which there appears to be a wide difference of

opinion is the proper thickness of the concrete section to be used in designing the shell. The author derives the formula, $T = \frac{pr - 9A}{12c}$, for the proper thickness of the concrete wall at any point, based on the assumption that vertical cracks are not likely to develop under a tension of 290 lb. per sq. in. in the concrete, and a ratio of 10 for the modulus of elasticity of steel to concrete. With the unit stresses assumed, the formula reduces to the simpler form that at any point the thickness of the concrete must be 42.7 times the area of the steel per inch of height of wall, thus limiting the steel reinforcement in the shell to 2.35 per cent. Mr. Potter.

The assumption on which the foregoing results are based appears to be an arbitrary one. There are any number of existing structures in which the ratio of steel to concrete exceeds 2.35%, and in which the concrete remains practically water-tight. Perhaps the most striking example can be found in cement-lined, wrought-iron pipe. In such pipe the steel reinforcement is frequently as high as 10%, and yet, under the high unit stresses to which the wrought-iron shell is often subjected in service, the cement lining possesses sufficient ductility to remain intact after years of service. Many sections of such pipe have been removed by the writer from distribution mains, the lining being found as perfect as on the day when laid. There are other structures, such as reinforced concrete pipe and stand-pipes, in which the ratio of steel to concrete in the shell exceeds 2.35%, which appear to maintain their water-tightness.

In a circular reservoir, the concrete in the shell performs two distinct functions: first, to render the structure water-tight; and second, to resist, in combination with the steel, all stresses due to bending moments which are likely to develop in the more or less rigid structure. The thickness of the concrete lining required for water-tightness, theoretically, is entirely dependent on the density of the concrete and the hydraulic head to which it will be subjected. Theoretically, therefore, it would be possible to use a very thin concrete section, provided the concrete was dense and water-proof, and the steel reinforcement very closely spaced. It is not found practicable, however, to do this. To build a truly circular reservoir is impossible. Owing to the lack of sufficient flexibility on the part of reinforced concrete, heavy bending moments are set up in the shell, due to irregularities of shape, and these must be resisted by the concrete in combination with the steel. It can readily be shown that, with only one row of circumferential reinforcing bars, the bending moments with their resultant stresses are very great and require a heavier concrete section than when two rows of circumferential reinforcing bars are used. The writer agrees with Mr. Flad that it is advisable to limit the percentage of steel reinforcement in the shell. The rule

Mr. Potter. given by him, therefore, based on an apparently arbitrary assumption, appears to be conservative, especially when two rows of steel reinforcement are used. With only one row, it may be advisable to limit the ratio of steel to concrete to 1%, especially in reservoirs of large diameter.

As most engineers are, perhaps, more interested in the general subject of water purification than in the details of reinforced concrete design, the writer takes the liberty, without fear of unduly prolonging the discussion, to remark on the tank from the operating standpoint.

To remove the settled solid matter from this settling tank, it is necessary to draw off the water and place the tank temporarily out of commission. This is likely to happen at a time when the character of the water is such that it will need treatment most, and when the entire settling capacity of the water purification plant is most needed for efficiency. Unless such tanks are cleaned out regularly, the sludge deposits will greatly reduce their settling capacity, thereby lowering the efficiency of the settling process.

The writer has always been of the opinion that a tank in which the settled suspended and precipitated matter can be removed at intervals, without interfering with the operation or efficiency of the tank, possesses considerable advantages over one which must be emptied and put out of service to remove the deposited sludge. Such a basin operating continuously has a number of advantages over one which is operated intermittently. There is considerable saving in the size of the settling basin when the settling process is carried on continuously. This saving may amount to as much as 50% over the intermittent installation in which two basins are used, decreasing somewhat with the number of basins. When the settling basin is operated continuously, its capacity is practically always available. This is not the case with the intermittent type.

There are a number of water purification plants in which the settling tanks are operated continuously. Without interfering with the operation and efficiency of the tank, the sludge is removed daily, and while it is still in a semi-liquid condition, before it has had time to compact. Settling tanks of this type have been used by the writer at McKeesport, Pa., and Georgetown, Ky. (both water-softening tanks), and also in the settling tank at Muskogee, Okla. In all these tanks, the sludge removal is accomplished by having the bottom of the settling compartments perforated with ½-in. to ¾-in. circular holes not more than 3 ft. from center to center in all directions. These holes discharge into an under-drain system leading to a common sump. Quick-opening valves are used to regulate the sludge discharge, and when such a system is well designed the possibility of the under-drains becoming clogged is very remote. All these plants are now in suc-

cessful operation. The oldest is at McKeesport, and has been in continuous operation since 1907. Mr. Potter.

The cost of properly under-draining a settling tank so that is can be operated continuously is not excessive. For instance, the Muskogee settling basin,* constructed of reinforced concrete, 212 ft. square, 18 ft. deep, and holding 6 000 000 gal., was built for a contract price of $45 000, including all piping, valves, and under-drains.

Charles B. Buerger, Assoc. M. Am. Soc. C. E.† (by letter).—The choice of a circular tension-ring type of tank, in this particular design, leads to the thought that there must be some economical dividing line such that tanks of smaller diameter are more economically built with a tension-ring wall, and those of larger diameter are most economical when built with cantilever walls. Mr. Buerger.

In actual practice the most economical style will depend largely on the particular details of design adopted, the unit stresses, the assumptions as to actions of the various parts, arrangement of metal, thickness of walls, the nature of foundation, and other varying data. There is, however, a general relation which can be established which shows what is for ordinary conditions the approximate maximum diameter for which the tension-ring type of tank is adapted.

Assume an ideal theoretical tank with tapered concrete walls of zero thickness at the top water level and a maximum thickness at the bottom, and assume that the active metal is in all cases 3% of the cross-section of the concrete, measured from the face of the concrete to the steel. Assume that the active steel is in all cases exactly equal to the theoretical quantity needed, with no allowance for laps or bonds; assume, also, that the steel at right angles to the active metal used to counteract shrinkage or temperature stresses, or to transmit stresses, is the same for the two types. For these conditions the only unequal element will then be the area or volume of the active steel.

Taking all dimensions in feet;

For the tension-ring type wall:

$$T = p\,r = 62.4\;hr, \text{ at any point } = f\,a \qquad (1)$$

where a = the area of the metal,

and f = the unit stress in the metal.

Calling H the total height of the tank,

$$A\,f = \frac{62.4\;H^2\,r}{2} \qquad (2)$$

and the volume of steel per foot of tank periphery is

$$V = \frac{31.2\;H^2\,r}{f} \qquad (3)$$

* The details of construction and methods of operation are described in the *Proceedings* of the American Water Works Association, 1912.

† Now M. Am. Soc. C. E.

Mr. Buerger.

For the cantilever-wall type:

As before, the total pressure acting on the wall above any plane

$$= 31.2\ h^2 \ldots\ldots (4)$$

The moment of the pressure at that plane,

$$m = 31.2\ h^2 \times \frac{h}{3} = 10.7\ h^3 \ldots\ldots (5)$$

For the assumption of 3% of metal and a ratio of modulus of elasticity of the steel to that of the concrete of 15, calling d the thickness of the wall to the steel

$$0.80\ f\ a\ d = m = 10.7\ h^3 \ldots\ldots (6)$$

$$0.03\ d = a \ldots\ldots (7)$$

$$a = \sqrt{\frac{0.40}{f}} \times h^{\frac{3}{2}} \ldots\ldots (8)$$

$$d\ V = a\ dh = \frac{0.40}{f} \times h^{\frac{3}{2}}\ dh \ldots\ldots (9)$$

$$V = \frac{2}{5}\sqrt{\frac{0.40}{f}} \times H^{\frac{5}{2}} \ldots\ldots (10)$$

For the dividing line, at which the required steel is the same for both types of wall:

$$\frac{2}{5}\sqrt{\frac{0.40}{f}} \times H^{\frac{5}{2}} = \frac{31.2\ H^2\ r}{f} \ldots\ldots (11)$$

$$r^2 = 0.000066\ f\ H \ldots\ldots (12)$$

$$\text{Taking } f = 144 \times 15\ 000 = 2\ 160\ 000$$

$$r^2 = 144\ H \ldots\ldots (13)$$

$$r = 12\sqrt{H} \ldots\ldots (14)$$

The general relation appears then that the maximum economical diameter of the tension-ring type of tank is 24 times the square root of the water depth. This relation will be modified in actual designs by several elements not considered in the foregoing. For instance, the quantity of steel to bond the cantilever wall into the floor of the tank has not been considered; and, as this is somewhat more in the cantilever type than in the tension-ring type, correction of this omission would be in favor of the tension-ring type. On the other hand, in the cantilever-wall type it is possible and economical to make the thickness of the wall greater than assumed in the foregoing equations, so that the percentage of metal is less than 3. The addition of this condition will favor the cantilever-wall type.

Applying this relation to Mr. Flad's executed reservoir, with the bottom of the wall at Elevation 97 and high-water line at Elevation 128, and a depth of 31 ft., it appears that the maximum economical diameter would be 134 ft. Mr. Flad's tank is somewhat larger than this, with a diameter of 150 ft.; but there are some conditions stated in his paper which amply explain and justify his choice of design.

The foundation is stated to be river silt, and is no doubt ill adapted to support cantilever retaining walls satisfactorily. It is obvious that the more uniform loading given by the tension-ring type of tank justifies the use of additional steel reinforcement. Mr. Buerger.

A. W. BUEL, M. AM. SOC. C. E. (by letter).—The author assumes the ratio of the coefficient of elasticity of steel and concrete at 10, which would seem to be low enough for any purpose, and sufficiently safe. Then, in the formula for the value of T, if the writer understands it correctly, the author seems to use a ratio of 9, making the tension in the steel, for 290 lb. per sq. in. in the concrete, only 2 610, instead of 2 900. Mr. Buel.

It would be interesting to know what the comparative cost would have been if, instead of proportioning the thickness of the concrete on the theory used by the author that vertical cracks would not develop if the tension be limited to 290 lb. per sq. in., the concrete had been made with a uniform and minimum thickness from top to bottom and with a separate water-proof lining inside. Unless there is a considerable saving in cost, the separate water-proof lining would appear to have the advantage of the other method, on the ground of many successful precedents.

EDWARD WEGMANN, M. AM. SOC. C. E.—Although not directly relating to the paper under consideration which the speaker has been asked to discuss, it may be interesting to describe the recent partial failure of the reinforced concrete Ambursen dam, built in 1912 and 1913 across Stony River in West Virginia, for the West Virginia Pulp and Paper Company. Mr. Wegmann.

In the fall of 1911, this company engaged the speaker's services as Consulting Engineer to examine the site selected for its dam, report whether it was suitable for this purpose, and recommend the type of dam that should be constructed. The dam was to be built on top of a mountain at the end of a log railroad. Owing to the great depth to rock, the cost of a masonry dam was prohibitory. There was no suitable material available for an earth dam, and therefore the speaker recommended that a hollow dam of reinforced concrete be built.

After a number of test pits had been dug and some borings had been made, four construction companies were invited to submit plans for the dam and bids for its construction. Before doing so, each of these companies sent a representative to inspect the site of the dam and the test pits. The plans submitted have been fully described.* Each construction company bid a lump sum for building the dam with a cut-off wall down to a certain assumed probable depth. Below this depth the cut-off wall was to be paid for by the cubic yard.

* *Engineering News*, September 5th, 1912.

Mr. Wegmann.

The dam was to be about 1 000 ft. long and was to have a maximum height of about 50 ft. above the surface. The lowest bid was \$143 000, received from Mr. F. G. Webber. The bid of the Ambursen Hydraulic Construction Company, of Boston, was \$189 000. In view of the fact that the latter company was the only one of the four bidders which had built a large number of reinforced concrete dams, the speaker recommended that the Ambursen Company be engaged to make the plans for the proposed dam. This was done. The contract for building the dam was awarded to Mr. Webber. To insure that the work would be properly done, the Paper Company engaged one of the engineers of the Ambursen Company to be constantly on the ground during the construction.

Mr. Webber began work at the west end of the dam and built about half the structure. The work was then taken away from him, as the progress he made was not satisfactory, and the dam was completed by the Ambursen Company at cost plus a certain percentage for profit.*

It appears that for about 200 ft. from the west end of the dam, the cut-off wall was founded on hard clay, and was made only 5 ft. deep below the floor of the dam. This shallow cut-off wall was built to where the water was from 30 to 35 ft. deep. For the remaining length of the dam, the cut-off wall was founded on rock. This part of the dam has remained intact.

The failure was caused by water finding its way under the cut-off wall, through a permeable, narrow seam of stone, sand, etc., that occurred in the clay formation. If the cut-off wall had been carried down a foot or two deeper, it would have intercepted this seam.

The water that passed through the seam showed itself in the weepers of the floor three or four days before the failure occurred. The watchman, left in charge of the dam, did not realize the danger this leakage indicated. It was only a day before the failure, that it occurred to him to inform the Superintendent of the Paper Mill at Luke, Md., about the leakage, which was steadily increasing in volume.

At that time a blizzard was raging and the temperature was 10° below zero. When the Superintendent arrived, the following day, the dam had been undermined and about 75 lin. ft. of the structure was destroyed.

The accounts of this failure which appeared in the daily newspapers were greatly exaggerated. The reservoir stored only about 800 000 000 gal. From the dam to the Potomac River, Stony River flows through a wilderness, the only building being a trapper's hut. The damage done, exclusive of the destruction of part of the dam, was very slight.

* The manner in which the dam was built is described in *Engineering News*, of January 22d, 1914, and in the same issue there is an account of the failure of about 75 lin. ft. of the dam, three or four months after the reservoir was filled.

Mr. Wegmann.

The failure was evidently due to the shallowness of the cut-off wall. In asking for lump bids for building the dam, the engineer of the Paper Company and the speaker had to assume certain depths to which the cut-off wall would probably be built. This depth was made 7 ft. below the top of the floor of the dam at the ends of the structure, 10 ft. where the depth of water was from 20 to 25 ft., and the wall was to be carried down to rock in the center of the valley. At each end of the dam sheet-piling was shown. The depth to which the cut-off wall was to be built had to be determined, of course, during the construction.

As actually built, the core-wall was founded on rock for about three-quarters of the length of the dam, and for the remaining length it was made only 5 ft. deep below the bottom of the floor of the dam. No sheet-piling was used.

The speaker's connection with the work ceased when the contract for building the dam was awarded, which was before the plans prepared by the Ambursen Hydraulic Construction Company had been received, and he had nothing to do with the construction.

Mr. Flad.

EDWARD FLAD, M. AM. SOC. C. E. (by letter).—Mr. Finch suggests that the working tensile strength allowed for the concrete, 290 lb. per sq. in., is excessive. He is correct in his statement that Mr. Andrews advises the use of this stress for 1 : 1 : 2 concrete. The writer is unable to state definitely at this date the line of reasoning that led to the selection of 290 lb. as a safe stress. Probably the original intention was to use 1 : 1 : 2 concrete, allowing the stress recommended by Mr. Andrews, and, later, when it was decided to use a 1 : 1½ : 3 mixture, it was concluded that the stress of 290 lb. would be well within the probable ultimate strength of that concrete; moreover an additional factor of safety was provided in assuming a high value for the modulus of elasticity of the concrete. In the structure as built no vertical cracks have been observed, but there is still evidence of moisture at some of the horizontal joints, due probably to a failure to clean the old surface properly before depositing the new concrete.

The construction suggested by Mr. Finch and Mr. Potter for the joint between the side and bottom of the tank appeals to the writer as an improvement, except for the difficulty of making this joint water-tight. In the design of the St. Louis Reservoir, the stresses at this point could only be determined approximately, there being a combination of cantilever action and hoop tension, and, in this respect also, because of the extra expense involved, the design is inferior to one containing an expansion joint at the junction of the side with the bottom.

The arrangement for the removal of sludge while in operation, suggested by Mr. Potter, would have been of doubtful benefit for this

Mr. Flad. particular reservoir, as there are two preliminary steel settling tanks, each 75 ft. in diameter and 25 ft. high, through which the water passes and from which the greater quantity of the sludge is removed before the water enters the concrete reservoir; there is no difficulty about arranging for cleaning the reservoir at times when it will not interfere with the operation of the plant, and the proportion of water to sludge required in cleaning is probably less than would be required with the plan of removal during continuous operation.

It is gratifying to note that Mr. Buerger's interesting comparison of designs containing cantilever walls with those providing for tension rings leads to the conclusion that the design selected, to wit, the tension-ring type, was practically justified, his theoretical determination of economical diameter for the reservoir in question being 134 ft., whereas the actual diameter is 150 ft. As Mr. Buerger surmises, the nature of the foundation available was really the controlling feature in deciding between the two types of reservoirs. A uniform distribution of the load was considered to be of paramount importance.

Mr. Buel seems to be in error in stating that, according to the formula given for the thickness of the concrete wall, the ratio of stress in the steel to stress in the concrete is 9 to 1. The formula is supposed to provide a ratio of 10 to 1, and is derived as follows:

Designating quantities and dimensions by the letters used in the paper:

Area of concrete per vertical foot of wall $+ 12\,T - A$,

Hoop tension $= pr = A\,s + (12\,T - A)c$,

$$pr = A\,s + 12\,Tc - A\,c,$$

$$T = \frac{pr - A\,(s - c)}{12\,c}.$$

Assuming that $s = 10c$,

$$T = \frac{pr - 9\,Ac}{12\,c},$$

which is the formula given in the paper.

The description by Mr. Wegmann of the construction and failure of the Stony River Dam emphasizes again the desirability of carrying cut-off walls down to rock or impervious strata. Happily, the failure was not as serious as at first reported, but it furnished a tender morsel for those who, through personal interest, were unfriendly to the Ambursen Company.

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Paper No. 1303

ROAD CONSTRUCTION AND MAINTENANCE.

AN INFORMAL DISCUSSION PRESENTED AT THE MEETINGS OF

JANUARY 23D AND 24TH, 1914.

PAGE

(1) ENGINEERING ORGANIZATIONS FOR HIGHWAY WORK.

By Messrs. Willis Whited, William H. Connell, E. W. James, Charles J. Bennett, A. W. Dean, Henry W. Durham, Robert A. Meeker, Paul D. Sargent, W. W. Crosby, Nelson P. Lewis, John C. Trautwine, Jr., William Goldsmith, George A. Ricker, William de H. Washington, Willis Whited, and William H. Connell.

Mr. Whited. Willis Whited, M. Am. Soc. C. E. (by letter).—This subject is of vital importance, because incompetency, inefficiency, or dishonesty in the handling of highway work may cause a larger percentage of waste of funds than in almost any other class of work. However excellent the personnel of the organization may be, there will be much waste unless all parties work harmoniously, steadily, and efficiently toward the end in view. There is the further difficulty, that, if the organization is not reasonably harmonious, many of the best men will become disgusted and resign, which would make it doubly hard to secure others to take their places. The subject can perhaps best be studied by taking a concrete example, such as the Pennsylvania State Highway Department.

The Department was first organized by an Act of the Legislature in 1903. The Act, however, was amended from time to time until 1911, when an entirely new one was passed, known as the "Sproul Act", reorganizing the whole Department, so that its functions may now be divided into three classes.

(1) The Department, at the expense of the State, is to improve and maintain, as funds become available, about 8 800 miles of roads, mostly main thoroughfares, set aside by the 1911 Legislature, to which the Legislature of 1913 added about 1 200 miles.

(2) The Department, under the law, is required to furnish to counties, townships, and boroughs, as funds become available, State aid for the improvement and maintenance of highways. The State pays one-half of the expense, the other half being borne equally by the county and township, or county and borough, as the case may be, the Department having entire charge of such road work.

(3) The Department, under a Legislative Act of 1913, has technical jurisdiction over all improvements on about 82 000 miles of road in addition to the 10 000 miles previously mentioned, and these roads are under the control of the Township Road Supervisors acting in their respective townships. The Department also has charge of the purchase of road-building machinery, exceeding a certain amount. Contracts pertaining to work of this class are not valid unless ap-

proved by the Department. The State bears one-third of the expense of such improvements. This places all public roads, outside of cities and boroughs, under the more or less complete control of the Department. Mr. Whited.

For handling such extensive enterprises, involving the expenditure of many millions of dollars annually, three methods may be suggested:

(1) The appointment of a commission consisting of three or more members, who shall formulate all general policies and appoint officers to carry them out;

(2) The appointment of a single head, with advisory council;

(3) The appointment of a single head, without advisory council, except such as he chooses to accept from his subordinates for special purposes, the Legislature itself determining the general policy of the Department.

The advantage of the first method is that the united knowledge and experience of the whole commission is available for the settlement of disputed points and questions of policy. The principal disadvantages are: first, the division of responsibility; second, it is apt to become a mere debating club, which seldom accomplishes much unless dominated by one man who, being an official, can with difficulty be prevented from abuse of power. A third disadvantage of this method is that a competent commission for such a purpose is usually composed of very busy men, who have many other interests, making it extremely difficult to get them together with sufficient regularity to settle questions with the promptitude necessary for the most effective prosecution of the work, particularly as their salaries are apt to be much less than the real value of their services.

The principal advantages of the second method are that the members of an advisory council usually have an adequate salary, and can be required to hold meetings regularly. On the other hand, being subordinates, they will have little power, and will therefore take but little interest in the work.

The third method, in which all authority is vested in a single head, constitutes in reality a despotic rule, which is by far the most efficient provided the proper man can be placed in the position and prevented from any misfeasance of office. The superiority of this form is shown by the fact that all military organizations from time immemorial have been under the control of single heads having despotic power, except that in modern warfare civil authorities have the right to deprive him of his commission in case he should abuse his powers or show incompetency. In warfare, efficiency is of paramount importance. Almost all industrial and business concerns are under the control of single heads, but with powers subject to reasonable limitations. The Pennsylvania State Highway Department is placed under a single head,

Mr. Whited. whose powers are limited by the ordinary laws of the Commonwealth and by the Act under which he is appointed, which designates the policy to be pursued by the Department.

All contracts are approved first by the Highway Commissioner and then by the Governor and Attorney General. All appointments to the higher positions are approved by the Governor. All expenditures are audited and approved by the Auditor General. Such checks as these should be sufficient to prevent the head of the Department from misusing his authority in making appointments and from the misappropriation of any funds.

Under the above-mentioned Act, the Commissioner is in absolute control of the administration of his Department, assisted by two Deputy Commissioners, also under his control.

The First Deputy Commissioner, in addition to assuming the duties of the Commissioner during his absence, is in charge of the Bureau of Township Highways, and also of the granting of permits for special uses of all public roads outside of municipalities.

The Second Deputy Commissioner, in addition to acting as an assistant to the Commissioner, has charge of the licensing of motor vehicles, and the enforcement of State laws relative to the use of such vehicles on State roads. He also has charge of the finances of the Department.

The Chief Engineer has immediate technical and executive control over the whole Department excepting the Automobile Division.

There is a clerical and auditing staff at headquarters, performing the duties usual with such positions.

There is an Engineer of Bridges, who has technical charge of all bridges and culverts.

There is a Superintendent of Signs, and a Superintendent of Asphalt Construction, whose duties need no description. There is also a Division of Tests, and a Division of Statistics, charged with the duty of obtaining data as to the traffic on all roads under the jurisdiction of the Department.

There are fourteen Assistant Engineers, who have technical and executive control over their respective Districts, but who act only in a technical capacity on township road work, and an additional Assistant Engineer acting as office assistant to the Chief Engineer.

The maintenance of State roads is in charge of a Maintenance Engineer, subordinate to the Chief Engineer, and in charge of fifty Superintendents, each of whom has charge of the maintenance of all roads in his respective district. These Superintendents, acting under the direction of the First Deputy Commissioner, also have immediate technical charge of township roads and bridges, and are required to approve all plans for road improvements to be carried out by the Township Supervisors, of whom there are about 6 500.

All bills are approved by the various heads under whose jurisdiction they have been incurred, and are then audited by the Department Auditor and the Auditor General of the State, thus rendering impossible any considerable misappropriation of funds. All payments on contracts must be approved by the Assistant Engineer, the Chief Engineer, the Department Auditor, the Commissioner, and the Auditor General of the Commonwealth. Mr. Whited.

All surveys and plans are made by the staffs of the Assistant Engineers, and checked by the Chief Draftsman and his staff, located at headquarters. They are then approved by the proper Assistant Engineer, the Chief Engineer, and the Commissioner. Plans for bridges and culverts are made and checked in the Bridge Division, and approved by the Engineer of Bridges and the Chief Engineer.

Thus it will be seen that all matters of importance are passed upon by at least two competent men, who are entirely familiar with the conditions surrounding each case.

All contracts are approved by at least three men, in addition to the Attorney General of the Commonwealth, and all payments on contracts are checked and approved by at least three men before finally being approved by the Auditor General.

All work done under contract is under the direct supervision of an Inspector on the work, who is thoroughly conversant with the details. Such work is approved by the Assistant Engineer and inspected generally from time to time by the Asphalt Superintendent and Engineer of Bridges, each acting in his respective capacity, and is also inspected by the Chief Engineer.

Thus a serious error in design or construction, or the loss of funds through misappropriation, ignorance, or criminal waste, is effectively prevented.

Furthermore, nearly all the work is done under the eye and at the expense of the public, of whom a very considerable number are more or less familiar with road building, and who, by appeal to the press, the Courts, or public officials, could cut short any gross incompetence, negligence, or dishonesty of employee or official.

The Act provides that funds for the improvement and maintenance of all highways, other than Township Dirt Roads, shall be distributed as equably as possible throughout the sixty-six counties of the State, which effectively prevents the Commissioner from showing any undue partiality toward any particular portion of the State. Hence there is always one man who can be held responsible for results in carrying out the orders issued by higher authority, and, at the same time, in the decision of important questions, there is always one of competent knowledge to whom he can turn for advice.

All appointments to positions in the Department are made by the Commissioner, and are not subject to other approval, except in the

Mr. Whited. case of a few of the higher positions, which are subject to the approval of the Governor.

It is important that the Commissioner be given much freedom in making these appointments, because, when men having considerable authority are scattered widely over the State, absolute loyalty to the "chief" is of the utmost importance, and can be obtained more effectively by personal appointment than by any system of civil service examination.

At the same time, as many of the positions require a high degree of executive ability, as well as technical skill and knowledge, it is important that the appointments be placed, as far as possible, beyond the reach of "political spoilsmen". For this purpose it has been decided that, in the future, as far as practicable, all the higher positions shall be filled by promotion, after careful examination into the candidate's technical knowledge and experience, as well as a careful inquiry into his previous record, in order to ascertain his executive ability, trustworthiness, resourcefulness, tact, and general ability to get the best results out of all with whom he comes in contact, whether superiors, subordinates, equals, or strangers, these qualities, at least for the higher positions, being really more essential than a high degree of technical skill or knowledge.

It is needless to add that the Commissioner himself must be possessed of a high grade of technical skill, as well as great business and executive ability, in order to accomplish in the best manner the results expected of him by those to whom he is accountable.

Mr. Connell. WILLIAM H. CONNELL, ASSOC. M. AM. SOC. C. E. (by letter).—About two months ago a search for literature on "Municipal Highway Engineering Organizations" was made by the Library force of this Society. This search covered the past 5 years, and only one article dealing with this subject was found. Papers, however, have been written relating to the routine of existing highway organizations, and this has also been covered in some public works reports, including organization charts for highway departments, sometimes termed "street departments"; and it is apparent from some of these papers and reports that the officials in charge are making the best of the conditions under which they are operating, but that they are simply outlining and dealing with existing organizations, without comment or constructive criticism regarding the organization of a modern highway bureau. This being the case, it may be well to pause for a few moments and analyze some of the reasons for paying so little attention to the organization end of this important subject, which is second to no other municipal engineering division, from a business and engineering standpoint.

Highway engineering is not new, and considerable advance, resulting in improved pavements, has been made within the past 50 years,

but this advance has been achieved by the efforts of a few engineers working under tremendous disadvantages, through lack of co-operation by the Engineering Profession and the fact that highway engineering has been considered seriously by engineers as a body only within the past few years. The highway work, in a large percentage of municipalities has been handled by all classes of officials from as many different walks of life (none of which gave them a claim to any qualification for the work), or by a city engineer who has busied himself with water, sewer, and bridge problems, the nature of which diverted his attention from highway work. The latter involves so much detail of an engineering nature that it is not apt to appeal to one who is wrapped up in the problems pertaining to other branches of the Profession. As a result, the highway work was allowed to drift along until the highway department was considered to be the property of the politician, and, still worse, only recently it has been used by some business administrations throughout the country to parcel out a few jobs to men probably deserving of some recognition; consequently, the highway bureau has been made to suffer through the appointment of men conspicuously unfit for the work. In this policy a great mistake has been made, as the highway bureau, in a large measure, is the principal show-case of the city government—the pavements representing the goods in the window—and it behooves any city or State administration, political or otherwise, to look to its highways, for the public is alive to the situation, and recent developments have proved that more people can be reached and satisfied through an engineering highway organization conducted on a high plane, than through any other branch of public work. An adequate organization, however, is essential, as a successful highway administration is dependent on conducting and controlling the work with the least friction. Ease of operation is the most important factor, and this can only be obtained through an organization commensurate with the demands on it. Mr. Connell.

The political view of the highway department was expressed only recently by a leading political light of one of the largest cities in America, who stated that the highway inspectors in his city were all right because "they used to be truck drivers and consequently knew the streets pretty well." This statement was made in all seriousness, and to an engineer. It is small wonder, then, that many highway organizations reflect little credit on our municipalities. In fact, it is remarkable that the few engineers who have devoted their lives to this great work should have succeeded in producing so many good pavements, for in these there has been very little improvement, as far as the standard types are concerned, within the past few years, or since engineers in general have become interested in the subject.

Much has been said concerning municipal and State highway organizations by new and inexperienced officials, who, instead of employing

Mr. Connell. experienced engineers to plan their organizations, attempt to do so themselves, and often write articles on the subject which only too frequently are misleading, as they are not based on good engineering judgment coupled with experience in management on a large scale, both of which are necessary in planning such organizations or writing on the subject. Of course, a man familiar with business organizations for controlling work can plan a highway or any other kind of organization, but not without the advice of one familiar with all the functions and work coming under the jurisdiction of the respective organizations. Therefore, if it becomes necessary to plan a highway organization, it is far better to have the services of an engineer versed in the principles of organization and familiar with the work. In other words, the most economical, the quickest, and the surest means of attaining the end in such an undertaking is to pick a man whose qualifications come nearest to fitting him for the work, and, in the case of highway work, select a highway engineer.

With reference to the organization itself, the writer will assume that he is dealing with a large city, and that necessary modifications should be made, depending on its size. In a small city, of course, it would not be practicable to have as many separate departments as in a large one, but the principles set forth can be followed in the handling of the work. Instead of discussing existing organizations, an attempt will be made to outline a proposed highway organization commensurate with present-day requirements. In this it is most important to determine what functions should come properly under the jurisdiction of a modern highway bureau and just where this jurisdiction should start. The first step, or the laying out of the highways, should come under the head of "city planning," which is becoming more and more a special branch of the Engineering Profession, and the work is of such a character and the field is so broad that obviously all municipal engineering departments will necessarily have to be more or less affiliated with the city planning division, and the head of this division might be termed the city engineer. Therefore, it will be assumed that the highway bureau, after the layout has been planned, will start with the design of all work pertaining to the highways, including parkways, park drives, and small highway bridges. The larger bridges should come under a bridge department, as bridge engineering is distinctly a special branch of the Profession. After the design, of course, all necessary engineering work relating to lines, grades, and inspection, from the grading to the finished pavement, should be under the jurisdiction of the highway bureau. Before leaving the subject of design—in view of the tendency of late in our municipalities to create a central bureau of design—it might be well to call attention to the fact that the engineers in charge of construction and maintenance of

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highways would necessarily become specialists in this line of work, and should control and be held responsible for the design as well as the construction and maintenance. This is simply the logical procedure which should be adopted in any such work. If, therefore, there is to be a central bureau of design—which may be very desirable—it should be under the city planning division, and the designing—as far as the construction of highways is concerned—should be supervised by the highway bureau.

The next thing to be considered is one which is becoming more serious every year, namely, sub-surface structures and encroachments, which include all underground conduits, pipe lines, vaults, etc., steps, street signs, stands, and every kind of encroachment beyond the building line. These matters, together with permits and licenses for automobiles, vehicles, etc., should be under the supervision of a branch of the highway bureau. With respect to conduits, pipe lines, etc., the only solution of the problem for the protection of the pavement, convenience of the public, and general traffic conditions, would appear to be pipe galleries; but, in large cities, which are already paved and built up, this would require an enormous outlay. This question should be considered seriously, however, in growing municipalities.

Having provided for the design, sub-surface structures, and engineering work, including inspection, the next subject is the surface of the street, and here the first thing to determine is just where the jurisdiction of this bureau should begin and end. Generally, in all municipalities, the city controls the highways from house line to house line, regardless of the division of expense for the footways, paving, repaving, etc., which varies in different cities. This is a big problem in itself, and will not be discussed at this time. Assuming, therefore, that the city has such control over the highways, encroachments, street signs, license fees for automobiles, vehicles, etc., naturally come under the jurisdiction of this bureau, and right here is a big problem—as is also the case with sub-surface structures particularly relating to vaults—that has not been solved, namely, an equitable and just rental value to be levied by the city for all kinds of projections—underground or above the surface—beyond the house line, and license fees on all classes of automobiles, vehicles, etc. This, however, as in the case of the division of expense for footways, paving, repaving, etc., is too big a question to consider at this time. To continue with the highway organization, it will be assumed that the supervision, control, and enforcement of all the regulations governing these matters come under the jurisdiction of the bureau of highways.

The next, and probably the most important, matter to be considered in connection with the work rightfully coming under the jurisdiction of a modern highway bureau is a branch of municipal work

Mr. Connell. which is closely related to highways and plays an important part in their maintenance, namely the cleaning of streets. This branch of engineering work—for it is distinctly engineering work, as the life of the pavements and roads depends to a considerable extent on the kind of cleaning they receive—has never, except in a few municipalities, been placed on an engineering basis, and in many cities this work comes under the jurisdiction of a separate department, which not only results in increased supervision and overhead charges in connection with the care of the highways, but an overlapping of jurisdiction with regard to the control. In a well-organized highway bureau, including street cleaning, the street patrol inspection can be and is taken over by the men supervising the street cleaning. This makes it unnecessary to have a separate street patrol inspection force, such as is employed in many highway bureaus. The street cleaning forces are constantly at work, covering every street in the city at frequent intervals, and, being supplied with a memorandum book or blanks for the purpose, can write down observations of the conditions of the streets as well as supervise the street cleaning work.

The tendency of the day is to standardize and control work in a manner which will not only conserve energy and do away with unnecessary duplication, as far as individual effort is concerned, but will concentrate forces so that operations can be carried on efficiently and economically along the line of least resistance, as ease of operation is one of the most important factors to be considered in planning any working organization, and no step would accomplish more in this direction than the combination of the street cleaning and highway bureaus. This would have the added advantage of placing this work more generally on an engineering basis, which would undoubtedly lead, not only to more sanitary methods of cleaning, but to methods which would be more apt to take into consideration the maintenance and care of the pavements themselves than is likely to be the case where the street cleaning is not under engineering supervision.

In connection with the street cleaning work there is also the collection of ashes and rubbish and the collection and disposal of garbage. Although this work is not distinctly a highway matter, the collection and disposal of ashes, and rubbish particularly, is so much a factor of the problem of preventive street cleaning—in which the crusade against overloaded and leaky wagons, the throwing of papers, store sweepings, etc., into the streets, plays an important part—that it more properly belongs under the highway than any other municipal department. In a lesser degree, this is also the case with the collection and disposal of garbage. The writer, therefore, for the time being at least, or until a better solution of this problem is presented, will place both these matters under the jurisdiction of the highway bureau.

With reference to the division and subdivision of the work, assum-

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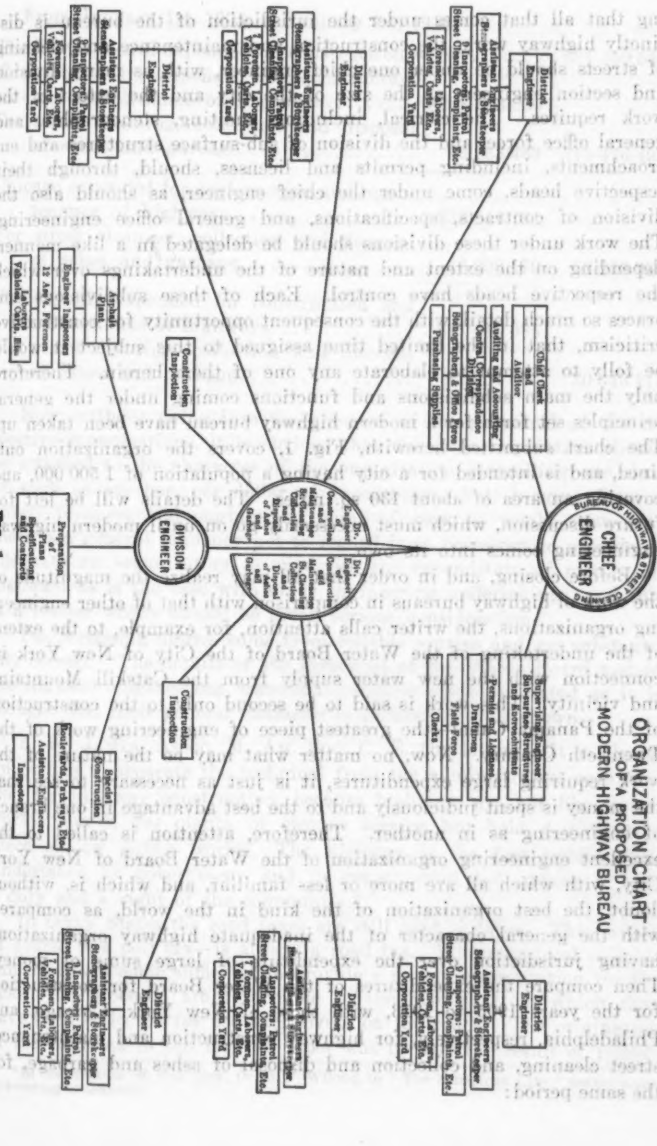

FIG. 1.

Mr. Connell. ing that all that comes under the jurisdiction of the bureau is distinctly highway work, the construction and maintenance and cleaning of streets should be under one chief engineer, with as many division and section engineers as the size of the city and the extent of the work requires. The clerical, including auditing, stenographic, and general office force, and the division of sub-surface structures and encroachments, including permits and licenses, should, through their respective heads, come under the chief engineer, as should also the division of contracts, specifications, and general office engineering. The work under these divisions should be delegated in a like manner, depending on the extent and nature of the undertakings over which the respective heads have control. Each of these subdivisions embraces so much detail, with the consequent opportunity for constructive criticism, that in the limited time assigned to this subject it would be folly to attempt to elaborate any one of them herein. Therefore only the main subdivisions and functions coming under the general principles set forth for a modern highway bureau have been taken up. The chart submitted herewith, Fig. 1, covers the organization outlined, and is intended for a city having a population of 1 500 000, and covering an area of about 130 sq. miles. The details will be left for future discussion, which must necessarily go on until modern highway engineering comes into its own.

Before closing, and in order that all may realize the magnitude of the work of highway bureaus in comparison with that of other engineering organizations, the writer calls attention, for example, to the extent of the undertaking of the Water Board of the City of New York in connection with the new water supply from the Catskill Mountains and vicinity. This work is said to be second only to the construction of the Panama Canal—the greatest piece of engineering work of the Twentieth Century. Now, no matter what may be the nature of the work requiring large expenditures, it is just as necessary to see that the money is spent judiciously and to the best advantage in one branch of engineering as in another. Therefore, attention is called to the excellent engineering organization of the Water Board of New York City, with which all are more or less familiar, and which is, without doubt, the best organization of the kind in the world, as compared with the general character of the inadequate highway organizations having jurisdiction over the expenditure of large sums of money. Then compare the expenditures of the Water Board for construction for the years 1905 to 1913, with those of New York, Chicago, and Philadelphia, respectively, for highway construction and maintenance, street cleaning, and collection and disposal of ashes and garbage, for the same period:

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From January 1st, 1905, to January 1st, 1913:

The Water Board expended on construction work........		$87 550 540
New York City expended for construction and maintenance of highways.........	$108 523 030	
For street cleaning, collection and disposal of ashes and garbage................	66 953 463	175 476 493
Chicago expended for construction and maintenance of highways................	$36 846 751	
For street cleaning, collection and disposal of ashes and garbage................	16 209 179	53 055 930
Philadelphia expended for construction and maintenance of highways...............	$17 445 085	
For street cleaning, collection and disposal of ashes and garbage................	12 904 030	30 349 115

Now, without going into the question of the personnel of the respective organizations, it is obvious that the same importance should be attached to the character of the organization controlling one class of work as another, per dollar of expenditure; but, judging from a purely business and engineering point of view, it would seem that there is a general tendency not to regard it as necessary to provide as adequate an organization to supervise the expenditure of monies on highways as is the case with other engineering undertakings, and this is a matter which should be given serious consideration.

The foregoing figures come from reliable sources and are substantially correct. The details of the personnel of the Water Board and of the highway engineering organizations have not been stated as it is thought that their general character is sufficiently well known.

Mr. James.

E. W. James, Assoc. M. Am. Soc. C. E. (by letter).—It is almost impossible to separate the technical from the administrative organization for highway construction, because highways are not like waterworks, sewers, gas plants, or electric service, but hold a distinctive place among public works. The highway is the first public work undertaken by government, and its construction and upkeep is both a duty and a right of the community.*

In many States the road taxes are still paid in labor. This, which of course is a poll tax—is always a road tax. Wherever local enlightenment has become sufficient to provide a money commutation for the labor, the proceeds are returnable to the district from which they have been collected for use exclusively on the roads. The origination of the road tax in the form of labor was in part the result of follow-

*In elaboration of this idea, Dr. D. F. Wilcox, in "The American City," makes some interesting comments.

Mr. James. ing the English system of local government, which alone was generally understood by a majority of the early settlers in America, and in part the result of the poverty in money resources and the richness in brawn and time resources which characterized newly settled communities.

It is rather strange that the labor tax has persisted in some of the States, but wherever it still exists, there is usually a local reason which is considered sufficient to explain adherence to so antiquated and inefficient a method. That the old law stands is no doubt due somewhat to the fact that though conditions have changed, the administration of the law has continued in the hands of unskilled men who have been able to see neither its defects nor any way to remedy them.* Its persistence, however, has caused one very pronounced condition of mind among the citizens of a large part of the country. Most men's grandfathers who lived west of the Appalachians built their own roads; later, their fathers built their own roads; and, consequently, they of to-day believe that they can build their own roads, and they leave the road labor tax on the statute books year after year.

This idea is much more prevalent than the casual observer will usually admit, but when we see that communities, in which the traffic has grown to such proportions and weight as to demand a much better type of construction than formerly, continue to depend on the labor tax and try to build roads without proper engineering, without proper financing, without even proper tools in some cases, we catch a glimpse of the inertia which characterizes a persistent idea. When the rural county citizen undertakes road work, he continues to believe that what his grandfather could do, he can do, and he, more than likely, dispenses with engineering advice, if the local good roads association does not force him to seek it.

This attitude of mind was embodied in the road laws of practically all the Southern and Western States until within the past 5 or 10 years, and is still reflected in many of them. The following States continue to collect all or part of the road tax in labor: Alabama, Tennessee, North Carolina, South Carolina, Georgia, Florida, Mississippi, Texas, Louisiana, North Dakota, New Mexico, Oklahoma, Arkansas, Montana, Kentucky, South Dakota, Colorado, Idaho, Illinois, Indiana, Minnesota, Nevada, Missouri, Pennsylvania, and Wisconsin.

Almost entire lack of engineering organization in connection with highway building in a large part of the United States has been the direct result of these conditions. Further, whatever efforts have been successful during the past few years in putting new statutes on the State books, none has succeeded in eliminating entirely the common

* "Road Legislation for the American State," by J. W. Jenks, Am. Econ. Assoc., Vol. IV, No. 3, May, 1889, p. 23.

idea that the farmer is as good a road builder as modern days can furnish. Mr. James.

The county and town system of road administration which obtains almost everywhere, places the construction and maintenance of roads in the hands of the county commissioners. In many cases, the commissioners have authority to divide the county into districts and appoint a supervisor over each district. Frequently, a penalty is prescribed for a man who, if appointed, refuses to act as supervisor. These men are furnished with a list of taxable polls in their several districts, and their duties include the collection of the road tax. They summon the road hands under the labor tax system, on the conditions fixed by law, and see that the legal time, or so much of it as may be required, is spent in working the roads. Those road hands who choose are usually permitted to pay out at a rate varying from 50 cents to $3 a day.* The supervisor collects the commutations, gives a receipt, and spends the proceeds on the roads in his district. His accounts usually stand like this:

Collected in road taxes....................	$787.50
Paid out for road work....................	787.50

Generally, a large part of the sum charged to road work goes, by tacit consent, as his perquisite, into the supervisor's pocket.

There has been a somewhat strong movement away from such conditions during the last 10 years, and we find many new laws drawn to relieve us of such ridiculous and wasteful methods of road administration. Still, the remedy has not been general, by any means, and there are some interesting clauses in the new laws. In the Alabama Road Law of April, 1911, for instance, the State Highway Engineer "may * * * be consulted by the county commissioners," and in cases where projects involving State-aid funds cost more than $3 000, "the highway engineer, with the consent and advice of the proper authorities in the county, may prepare plans and specifications" for executing the work by contract.

In the Arizona law of June, 1912, it is declared that "all roads and bridges, when constructed, shall thereafter be maintained and improved when necessary, * * * under the joint auspices and direction of the State Engineer and Board of Supervisors, * * *."

The Virginia laws of 1904, 1906, 1908, and 1910, are a respectable effort to do better things, but, from the limitations placed on the constituted road officials, it would seem that the General Assembly believed that the county commissioners knew more about road engineering than the State engineers whom they might employ under the law. Iowa has recently passed a new road law (April, 1913) which is a departure in legislation. It is the most systematic effort to correlate

*The common rate is $1 per day, but it ranges from 50 cents in South Carolina, to $3 in Nevada.

Mr. James. road work done at county expense that has yet been made. It could well be studied by the legislatures of three-fourths of the States. The new State of New Mexico goes one step farther in its road law of 1912. A State commission is created, and one of its duties is to appoint all county road commissions, each of which is composed of three members.

In general, however, except where State funds are involved, there is no engineering supervision required over county road work in the rural sections of most of our States. The counties thus conditioned spend, according to the most accurate reports attainable, about $61 000 000 annually on their roads. This is 40% of the annual road revenues of all counties.*

As soon as a State organizes a highway department and sets aside funds to aid the counties, the first systematic engineering organization is developed. The types of organization are simple and rather uniform for most States. They are confined to two common systems: one has a corps of assistant engineers reporting directly to the chief engineer; the other has a group of division engineers reporting directly to the chief. In a few cases, notably in Ohio and New York, there are deputies having general charge of a department of the work, as construction, bridges, maintenance. In these cases division engineers report to their respective deputies.

Under the division engineers there are resident engineers and inspectors assigned to projects.

A review of these actual conditions presents several questions well worth discussing. It is obvious that much of the reluctance on the part of poor counties to seek engineering advice is because there is no man available in the county, and an outsider—besides being an outsider, which frequently is considered more or less objectionable in itself—cannot be secured at a compensation warranted by the county revenues. As we see, therefore, it is not until the State steps in and gives the county funds that there is any engineering supervision of a large part of the county work. Cannot some simple engineering organization be created to deal with this situation?

At a hearing of a legislative committee in a Southern State, the writer had an opportunity, about 2 years ago, to suggest a plan that he still believes has merit. The plan comprises a highway commissioner, paid by the State, drawn, if possible, from the staff of the State university or mechanical college, so that the additional salary would be small; a corps of division engineers assigned to groups made up of from eight to ten counties.

The commissioner would have general supervision of the division engineers, and they would have charge of all road work in the counties

*Cf. Jenks, *supra*, p. 43. The principal exceptions are Massachusetts, New York, New Mexico, and Iowa.

of their divisions. Many of these counties never would undertake macadam construction. Their roads, because of lack of local funds, must remain earth, sand-clay, or gravel for many years to come. The duties of the division engineers would be to supervise the expenditure of the county funds. The commonest needs in most rural counties are: the substitution of some material besides short-leaf pine for box culverts, the construction of adequate ditches, the proper crowning of the traveled way, direct and regular alignment, the intelligent mixing of sand and clay, the drainage of spring holes and swampy places, and the removal of vegetation from old mudholes. The division engineer's duty would be to see that suitable construction gangs are organized to work over the county and do their work properly. Labor taxes should be commuted by cash payments, and the money used to pay the road gangs. The old supervisors should be discontinued, and a group of foremen substituted, who, if possible, should be retained permanently, and trained in their work. When bridges or culverts are needed, the division engineer or commissioner can furnish plans, and the division man should see that the work is done in a correct and efficient way.* Mr. James.

This plan would much reduce the cost of engineering to the counties by a general sharing of expenses, and would be productive of good results in several directions. It would do away with the supervisor system, and would tend to remove the work from political influences. It would concentrate expenditures, and prevent the general custom under the ordinary county method of working every road in sight to please as many voters as possible, with the obvious result that the work is spread out so thin that the rainy or winter season destroys it all. It would create some standardization and begin the elimination of hopelessly ineffective methods used year in and year out by the county officials without apparent betterment of the roads. It would create in time some sentiment favorable to the employment of private engineers and a general raising of the standard of all public works. This latter circumstance would be a most happy result, for the need of adequate engineering in small municipalities throughout the South and West, and indeed almost everywhere in the country, is glaringly apparent on the most casual investigation.

When State funds are appropriated, conditions are at once produced that direct attention to another question. Shall the organization distinguish between construction and maintenace? The last report made by the New York State Commission recommended increasing the number of divisions and combining the responsibility for maintenance and construction. This recommendation was interesting and significant, coming from an organization which has had such a varied history as

*Jenks, *supra*, p. 48.

Mr. James. the New York State Commission. The State Legislature, acting in part on this recommendation, passed a law permitting an increase from six to not more than nine divisions, and otherwise embodying the recommendations. The type of commission was also changed, and a single responsible head provided. The present highway department in New York works under this new statute.

The labor organization and type of inspection required on construction, on one hand, and on maintenance, on the other, are so widely diverse that it appears to the writer that some separation of the two classes of work can be advantageously made. In the first place, if the division or district heads do not have to split their duties between the two classes of work, each can cover a larger territory. Further, if there is a system of standards used, the maintenance divisions can be enlarged beyond the size of those possible for construction divisions. This is so because, in any system so large as to demand division or district organization, the quantity of work makes construction by contract practically compulsory, and contract work requires constant and detailed inspection. On the other hand, maintenance by either the patrol or gang systems is force account or administration work, and as the patrolmen, foremen, and gangs are trained, they require less and less minute supervision. As a further consideration, it is not a bad plan by any means to establish as soon as possible the separate identity of the maintenance organization; because at last this will be the only one left, as in the case of the French *Routes Nationales* today. Trained engineers are generally required to take residencies, whereas a division, when entirely under maintenance organization, would probably require no such resident engineers. In very large maintenance divisions assistants might be necessary to handle the larger re-surfacing projects, but in most cases trained foremen would be able to do a large part of the work under general orders.

On the other hand, an indisputable advantage comes from the vertical organization of divisions. If an engineer has entire charge of a restricted mileage during construction, he is doubtless the best prepared man obtainable to handle the maintenance of the same sections. He has watched the construction and knows the weak spots, the bad soil, the softer stone, and the doubtful drainage arrangements. He knows the county thoroughly, and should have developed amiable relations with local officials and the citizens along the roads. All these matters would count much in a man's equipment for division work, but there is no insurmountable reason for a division man not leaving record maps sufficiently complete to inform a successor, instructed to take over roads for maintenance, in all the major and many of the minor conditions along the sections under observation. This matter of small-scale record maps is capable of large and valuable development. Further, a man detailed to maintenance, and concen-

trating his attention on it, should soon acquaint himself with the conditions in his division. It is seldom that labor used in construction stays to maintain, and, therefore, no especial advantages in the line of securing skilled laborers can be expected under either system. A new labor and supervisory organization will generally have to be created, and, in most instances, be recruited from the local supply. Mr. James.

Another matter that, from the standpoint of efficiency and economy, is of especial importance, is the place occupied under the State organization by the county road administration. Three States, New York, Iowa, and New Mexico, have statutes specifically placing the county road engineers or supervisors in the State system. There is no denying that this innovation (for, in the present condition of our State road laws, it can certainly be considered such) has little about it to be unfavorably criticized. Of course, when within a State the question of local autonomy comes up in any form, we always find some who oppose any restriction on home rule; but, if we analyze the basis of home rule, we find it chiefly concerned with matters of conduct. Now, road building is not a matter of conduct. Our fellow-citizen does not understand why his road money should not be economically expended by competent engineers. Now that two or three States have led the way, it may be hopefully expected that there will be a further move in the direction of placing some check on the indiscriminate expenditure of county funds under a local system which is worse than simply inefficient. This can be done, even when no State-aid feature enters in as a sop to the county. When such allotment does appear, there is no reason why supervision should not extend beyond the limits of the State-aid roads.

In Iowa the county engineers and the county officials, as far as road matters are concerned, are under the supervision of the State commission. In New Mexico the county road commissions are appointed by the State commissioner. In New York, county and town supervisors are under the division engineers, and in Illinois, the county engineers are appointed from a list of eligibles satisfactory to the State commission.

Just how far the State organization shall be indicated by law is a matter of debate. The writer believes that few States err on this point at the present time. Generally, except in three or four cases, the detail of the highway department organization is not carried beyond the State highway engineer under various titles. The details of organization are left to the administrative heads and the engineer. This is as it should be, until we find some organization that has details of particular merit capable of wide application. This freedom, however, does not preclude placing the county road authorities by law under the supervision of the State organization.

Mr. James. Many of the State departments to-day are advisory or nominally supervisory in their duties, and it is this restriction which, in spite of freedom in departmental organization, restrains their usefulness in so many cases to almost negligible limits. Where the great gain in efficient expenditure must come is in the county work, and the State engineer is helpless, however competent he may be, unless he is given some element of authority over the local activities. Merely advisory work is very often gratuitous. Some county officials consider it almost meddlesome. It is these local Solons that good organization must circumvent.

A review of the several State laws shows that the highway departments suffer from one very general condition. There are few single responsible heads. We are familiar enough with actual working conditions in some States to know that the work suffers, or has suffered, seriously because responsibility is not concentrated in one administrative official. It is very common to have the law stipulate that the engineer shall work in conjunction with an administrative official, sometimes even with an administrative body, of equal or greater authority. Arizona presents a case of this sort. In some States work is being done successfully, because, with a poorly devised law, a commission of three or five members has turned the work over to one of their number, or to a chief engineer, who acts practically as a single head. This has been conspicuously true in Virginia. The ill success of a many-headed commission can be seen in Rhode Island. There a commission of five members, one from each county, has had active charge of work, and instances have occurred where commissioners, visiting construction in progress, have changed the orders of the engineer in charge.

With regard to details of organization under the commission or responsible heads, the laws in most cases leave the highway department free to adopt such system as seems best fitted to accomplish the ends and purposes of the statutes. Were there a general system of single responsible heads, many States under their present laws, could develop thoroughly efficient organizations. The responsibility for the organization would lie with the head, and he would generally be unrestricted. There are thirty-eight States which have some form of highway department, but it is doubtful if more than fifteen of them can develop efficient highway organizations, because of the divided authority and responsibility residing in several heads.

Mr. Bennett. CHARLES J. BENNETT, M. AM. SOC. C. E.—With one or two exceptions, the speaker is impressed with the general soundness of the arguments presented by Mr. Connell. In the first place the formation of a standard organization for a highway department in a city is not entirely feasible. There are so many difficulties in the way of standardizing any particular department of a city that it would be impractical to

consider any more than general principles to be used in all cities, instead of trying to standardize a system for one department in all. Mr. Bennett.

For instance, a standard system applicable to New York City, Chicago, or Boston would certainly not apply to a smaller place, on account, not only of the size of the city itself, but the general geography, characteristics of the people, and the system of government. It seems plain, therefore, that if a standard system is to be devised, all cities in the United States should be classified and a system of organization for all departments formed, so that the entire operation of city government could be along standard lines for different types of cities.

This seems to be an ideal which cannot be reached, and the problem, therefore, must be to apply general principles to engineering organizations which shall tend to improve conditions which are bad at present. These applications should also furnish engineers with information which can be classified by engineering societies, so that costs and systems may be contrasted and certain valuable improvements in results gained.

There is no question of the need of an engineer to supervise not only highway departments, but all public works of any character. Therefore, a Municipal Highway Engineering Bureau should be a part, and a very large part, of an Engineering Bureau of Public Works, and public works of all kinds should be under the supervision of a trained engineer.

This principle, if applied to cities, will work with incalculable benefit, both in the improvement of the construction of public works and in their economical operation after completion. Granting this, therefore, Mr. Connell's statement as to the need of an engineer to supervise all highway work is evident, and the supervision of this work, as he states, should include, not only the actual construction and operation of these public works, but also the design of the different structures as well as the outline of the general plan to be carried out in the expansion of the city.

Now, a word as to the qualifications of an engineer in public service, and the difficulties of securing one who is competent to follow up work of this class: Engineers by training should be fitted to do all things necessary for the operation of such a magnificent department as has been conceived in control of the public work of a city or State. That means that the engineer must be not only a technical man but a business man who can see both the scientific and financial sides of the problem, and determine the advisability of spending any given sum of money for a certain purpose.

The second qualification means that the engineer must take a broader view of public work than he generally does.

In considering the operation of any city or State engineering department (and the ideas here outlined apply, it seems, particularly to

Mr. Bennett. the highway department), there is need of a better training for engineers in keeping accounts and supervising routine office work. The success of any highway bureau, or any other bureau which is under the supervision of an engineer, depends on its accounting system as much as on any part of the whole situation. The expenditures must be kept within appropriations, and work must be planned and executed in such a manner as to gain the full benefit, so far as possible, from the expenditure of moneys allowed. Therefore, the engineer should be trained, and highly trained, along business as well as technical lines. The natural tendency would be for an engineer to be too much given to minutiæ and complicated methods. The endeavor should be to devise a system of accounts as simple and free from unnecessary complications as possible.

This discussion has followed general lines more than the particular ones specified in the topic, but these ideas—which must be considered as general, rather than specific—can be applied to a highway engineering organization.

In connection with the discussion on highway work, both municipal and State, it is interesting to note the diversity of opinion among engineers as to the value of the work done. Within the past year a prominent engineer has told the writer that there are very few engineering problems to be faced in highway work; that this work is of the simplest nature, and, by inference, almost any one could become a highway engineer.

To the ordinary man's mind, the highest type of engineering would be that which would do, in the best manner, both the simple and the complicated things.

There are comparatively few very large structures or projects to be carried out by engineers, but, as indicated by Mr. Connell, the largest amount of money spent in engineering work is for the operation of municipal and State highway departments, and the results of this work are open to critical observation by all who are in possession of five senses.

Although the engineering problems may be simple, the problem of spending public money in an economical manner, is, by far, the hardest one which the engineer has to solve, and if this can be solved by improved business methods, as well as improved technical methods, it seems that we will have done some of the highest class of engineering work.

In conclusion, the speaker endorses Mr. Connell's statement, that the highway or other departments of cities or States should be freed from political influence, and that the spoils system should be entirely eliminated in the operation of these departments. Only a moderate amount of success can be attained when any system of politics, whether it be good or bad, is allowed to interfere with a municipal or State

department, and when these departments are entirely free from political interference, then, and then only, will ultimate success be gained. Mr. Bennett.

A. W. DEAN, M. AM. SOC. C. E.—Very recently the word "efficiency" has become a particular favorite, not only with those who are directly concerned with State and municipal affairs, but with all who are interested in the administration and execution of public works. Witness the large number of governing bodies which have created commissions or boards of efficiency and economy to analyze and criticize the acts of public officials, and, by their advice, minimize waste and extravagance in the expenditure of public funds. Mr. Dean.

No department in State or municipality lends a greater field of action to such a board than the highway department, when competency is not the prime requisite in the selection of heads and assistants; and, on the other hand, no highway department consisting of competent men selected for their particular fitness for the positions they occupy will furnish a fair-minded efficiency board with any cause for reprimand or public criticism.

The preceding statements are made only to emphasize the fact that no highway organization can be successful unless competency is the first and only consideration in the selection, not only of the head, but of the entire body of the organization. Competency and efficiency are inseparable terms, for without one the other cannot exist.

A detailed description, either in words or graphically, of an organization adaptive to any and all conditions is obviously impossible. Highway laws in the several States vary widely, as do also conditions to be met in municipalities of varying wealth and population.

Supported by laws prescribing a well-defined policy in the expenditure of State funds, a single responsible head is preferable to a body of three or more; and, on the other hand, if the State has not, by legislation, fixed a general policy, a body of three may rightly be considered preferable. With or without a prescribed policy, a single responsible head with an advisory council having limited powers should prove efficient in carrying out large or small undertakings.

In a municipality it has been well demonstrated that a single responsible head is sufficient.

No organization can be long successful unless it is arranged below the head in military order. If the organization is small, the head may act as the chief executive; if large, there must be one chief executive charged with carrying out the general policies and orders of the head, as a division of responsibility at this point tends toward duplication, uncertainty, and consequent inefficiency.

Divisions and sub-divisions below the chief executive officer are dependent on the scope of the work to be done; in State highway organizations a suitable number of division engineers are charged with

Mr. Dean. responsibility for both construction and maintenance in their several divisions. In municipal organizations each division should include the entire municipality, with a division engineer in charge of each branch of the work, that is, one in charge of bridges, one for street construction, one for maintenance, etc. For large undertakings the division may be subdivided into districts, with district engineers in charge responsible to the division engineers.

To summarize and further present the matter briefly, the following outline of highway organization is submitted:

1.—A single commissioner, or three commissioners, as quite definitely recommended above;
2.—A single responsible chief executive officer;
3.—As many divisions as may be necessary, each under a division engineer;
4.—Sub-divisions or districts, each in charge of a district engineer;
5.—Minor assistants.

All appointments should be governed entirely by fitness; there should be no changes in the personnel, except for proper cause; there must be no division of responsibility; all orders must be transmitted as in military practice.

Mr. Durham. HENRY W. DURHAM, M. AM. SOC. C. E.—It is of interest to have confirmed by Mr. Connell's search, the fact, apparent to all who have been charged with city street maintenance and repairs, that there exists little constructive criticism based on broad lines as to the methods of carrying on such work. Destructive criticism abounds.

Reports of citizens' committees denouncing indiscriminately pavements of bygone days, or the non-use of some special, possibly patented, cure-all street surface, the promoter of which has had an accelerator at work, illustrated by views of the depression where the prominent citizen's automobile was bumped, and accompanied by half-baked legal recommendations based on the American citizen's ineradicable impression that the millennium can be attained by "passing a law"—these, and the annual reports telling what has been done with the tools we have at hand, and the textbooks that tell much about good pavements, but little as to paving organizations, comprise most of our highway literature.

There have been, it is true, studies made and plans advocated by efficiency experts for the better handling of the routine work of existing highway departments, or for their improvement in details, but the results of Mr. Connell's investigations would seem to indicate that little has been written on the fundamentals of organization and the proper scope required of the department in charge of highways in a great city. This is possibly due to the fact that in America the de-

mand for improved street surfaces, caused by the increase in number, weight, and speed of vehicles using the highways, is coincident with the arrival of conditions causing their upheaval to a maximum degree. In no other part of the world at the present time, and in no previous period of the history of the United States, has there been anything paralleling the amount of rebuilding and shifting of what were supposed to be established trade and social centers, that is to-day going on in all large municipalities. Mr. Durham

While the general public is demanding the construction of streets with absolutely smooth surfaces for availability for motor race-tracks, that shall be unyielding and without surface wear, so as to endure for all time, constructed over subways and pipe galleries to avoid all future surface disturbances, it is at the same time abandoning whole districts devoted to residences and light traffic, and filling them with tall office buildings which, when the expected office occupants fail to materialize, are turned into factories having a population in some sections of New York larger than the great mill towns of New England; and is demanding that these buildings have adequate service for telephone, electric light and power, and water supply, including high pressure for fire protection; and, finally is planning new lines of subways to carry away the surplus population brought to these districts, and to open up new possibilities of change. To harmonize these conflicting demands would appear to be an impossibility. It is under such conditions that officials charged with the maintenance of municipal highways are to-day carrying on their tasks.

To construct and maintain pavements under present-day traffic is an occupation in itself. When to that is added the fact that in no great city can the highway officials determine with certainty whether the pavement laid to-day will be suitable for the traffic to be found using it 5 years hence, it will be seen that they are entitled to the benefit of the best constructive criticism available, and that the fault-finder, who devotes what knowledge he may possess to enlarging his own newspaper reputation by the cheap method of criticizing obvious defects, is actually blocking progress.

Mr. Connell has taken up the problem in the right way, and has indicated an outline for the commencement of efficient highway organization. It may be of interest to have the subject also treated from other standpoints. In the course of an investigation of municipal highway work in European cities, made during the past year, some points were noted as to their methods of solving this problem.

Two general methods are followed in different parts of Europe:

1.—A strongly centralized authority under which the entire city is sub-divided into a number of smaller unit organizations, each within its district having charge of all municipal work, closely governed by the central office;

Mr. Durham.

2.—Local control where, in the city as a whole or as sub-divided in each of a number of independent units, a single head has control of all city work, with sub-divisions, each separated and independently charged with a single class of work.

The first system is best exemplified in Paris. In the organization of the street service and for all construction work in the highways, it is divided into seven sections, each headed by a resident engineer who has control, not only of paving work but of the laying of water pipes, sewers, and the supervision of any sub-surface work of private corporations. At the head of the bureau of street service is an Inspector General with necessary deputies and office organization, having under him several chief engineers, one in charge of public streets and street lights to whom the resident engineers primarily report; another at the head of the work of street cleaning; one in charge of gas and electric light mains; the fourth in charge of water and sewers.

Each chief engineer has the necessary office force, and several resident engineers are under his orders for special work and for the management and maintenance of work outside the city; but, within the city, all construction work on the streets is under the direction of the street service engineer in charge of each district. Each in his own section takes care of the paving work; lays water pipes; builds sewers; and supervises duly authorized construction on the part of gas and electric companies, so that no pavement can be torn up except by his orders or with his permission.

In London, on the other hand, the control is almost entirely decentralized. There each of the thirty different boroughs has its own city engineer or surveyor responsible only to the Borough Council for orders and directly in charge of all street paving, repairs, cleaning, and lighting. The works of general utility to the entire city, such as water supply, main-drainage sewers, and municipal tramways, are controlled by special boards independently of the boroughs, and do any necessary street work under the supervision of the local borough authorities. Smaller English cities have usually a city engineer, charged with control over all the city works, having under him a Department of Highways headed by a road surveyor, and an organization of assistants whose duties are confined to the maintenance, repair, and cleaning of highways. With some modifications, the system in the German cities is very similar.

In the first of the two systems there is one responsible head over all classes of work carried on in each unit—the unit being some convenient sub-division of the entire city—and though the different divisions of city work are each under special organizations, the actual execution, in theory at least, is controlled so as to render lack of co-operation or interference of different classes of work impossible.

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In the second system, in the city as a whole, or in each of its independent units, in the case of large cities, there is centralized control to the extent that all construction, with the exception of water-works, is under the direction of a city engineer, but the special varieties of street work are usually carried on by independent sub-units. Of course, many exceptions are found in any necessarily brief and general classification like this. For thoroughness and theoretically perfect, systematic, and orderly procedure, the French method of organization seems to leave nothing to be desired.

The city work of London, on the contrary, is carried on under what is probably the most illogical and unsystematic arrangement that could be devised, but, judged by practical results, London presents probably the best appearance of any city in the world, as far as the streets are concerned, and Paris is constantly in a torn up and poorly maintained condition.

In Berlin, a city about equal in size to Paris, but divided into a number of independent city or borough governments, without a central control, but very thoroughly organized as units, the results show good conditions in some districts and poor in others. The practical deduction to be drawn from the comparison of these methods, not on paper but as worked out in the three great European cities, is that no system will produce results automatically, and that good results can be produced under adverse methods by a people accustomed to orderly procedure.

It will be seen that the scope of a department in charge of public highways can be very broad or very much restricted and still be in accordance with good practice in some prominent city.

Mr. Connell's plan seems to follow the French system, but without going far enough for completeness. If the work of the street department includes cleaning, it may well go farther and take in sewer, lighting, water and other city works requiring the use of the streets. If we believe, like Sterne, that "They order this matter better in France," we may well consider that a model city works department should be based on that of Paris, which, after co-ordinating all possible interests, furnishes ample opportunity for the card-index and loose-leaf expert to devise interlocking systems to his heart's content until he arrives at his Utopia, where the Inspector Generalissimo, by a system of push-buttons at his desk, can run his department and never see the work.

The weak points which make this system difficult of application lie in the question of personnel. We have in the United States no trained class of public works officials, no body of Government engineers organized like the French Corps of Bridges and Highways, and we have frequent elections and still believe at heart that "to the victor belong the spoils" including the right of spoiling the work

Mr. Durham. done for us by the vanquished. It is hard enough to find one civil service man to fill a responsible position; to find seven equal and uniform would be almost impossible. In the light of possible conditions, it would seem to be a better policy to concentrate along all classes of work, organizing the Bureau of Highways into sections for doing specific things in charge of the men best qualified to do them. An outline of such an organization, based on experience in Manhattan Borough, but making improvements in some existing conditions, is somewhat as follows:

The head of the Bureau of Highways has charge of all construction and maintenance work on street surfaces between building lines, and issues and controls permits for necessary street openings of whatever nature, under the direction of a Commissioner of Public Works whose authority includes, besides highway construction, the sewers and other sub-divisions, and in whose office all financial matters, including auditing, contracts, etc., are taken care of.

To avoid duplication of work, a single Bureau of Design and Survey carries on all necessary work for the different construction bureaus in the department, its work for each, however, being under the direction of the head of that bureau. This applies, not only to the making of preliminary and contract plans, but to the furnishing of parties for surveys, giving grades, and making measurements for final estimates.

Under a Chief Clerk is the control of all correspondence and records of the bureau, including the necessary organization for filing, recording, and handling data for the maintenance of a live record as to street conditions, necessary repairs, etc.

As has been pointed out, the control of street openings is an essential preliminary to the work of any highway department. No system, however complete, will be of more value than as a historical record unless some plan lies back of the causes necessitating such openings. It will be essential ultimately that our great cities realize the desirability of the adoption of a systematic plan for sub-surface structures for new districts and take up the problem of an orderly re-arrangement of these structures in the most congested sections. Thus far, charter rights of individual companies and the unwillingness of various departments to co-operate with one another have rendered futile any attempts in this direction, resulting in a tangled mass of sub-surface structures in our central districts, with locations frequently unrecorded, and the repair or extension of which requires an undue amount of street surface disturbance. Pipe galleries under all streets are a dream of the idealist, but an orderly planning which shall place under footways the lighter house-service lines, leaving the large mains under the roadway at regulated intervals, and shall do away with unnecessary duplication of facilities in any one street, and

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a limited provision for pipe galleries in the most crowded sections is not impossible of attainment. It will require, undoubtedly, amendments to the charter, both of municipalities and corporations, and the vesting of the right to order changes in the body controlling the city policy. With such a condition existing, the permit office of the Bureau of Highways will have actual and not merely (as at present) nominal control over all street openings. This can be done without interfering with the rights of any other corporation or corporations to get at their structures for necessary repairs. The usual procedure will involve application to the permit office in advance of making any opening in the pavement, and a deposit of sufficient money to cover the cost of restoration, inspection of subsurface work, back-filling, and subsequent repair and maintenance.

After street openings are properly controlled, the making of repairs necessitated by ordinary wear is of first importance. For convenience, this work has been placed in two divisions: one having charge of all repairs to block pavement, including stone block, wood, and asphalt blocks, a large part of which work can be done by gangs with a limited quantity of new material, and repairs to sheet-asphalt pavements, which require the use of an asphalt plant either owned by the city or operated by contractors. To lay out the work of these divisions properly, a system of street inspection is essential, and there is planned a division of patrol inspectors, each having a definite district and making daily reports of all defects, which are immediately plotted on an office map from which the day's work for the gangs can be laid out to the best advantage. Another division looks after all matters pertaining to sidewalks, whether carried on by the city or the property owners, and has control of such other matters as street signs, removal of incumbrances, etc.

Inasmuch as the perfect non-wearing pavement has not, as yet, been discovered, it becomes necessary to provide for renewals of street surfaces from time to time. This, for the present at least, is best done by contracts, under the supervision of a division engineer with a necessary organization of assistant engineers and inspectors.

Whether the repair work is performed by city forces or by contractors, the purchase of new material and their conformity to specification requirements is essential. Hence, one of the most necessary divisions is the laboratory and plant inspection force in charge of the chief chemist. Losses due to mistakes or inadequate inspection in this division may amount to many times in value what would occur from omissions or carelessness of inspection on construction, and the city will save many times the cost of a proper organization to supervise the manufacture and delivery of proper materials of all classes for pavements.

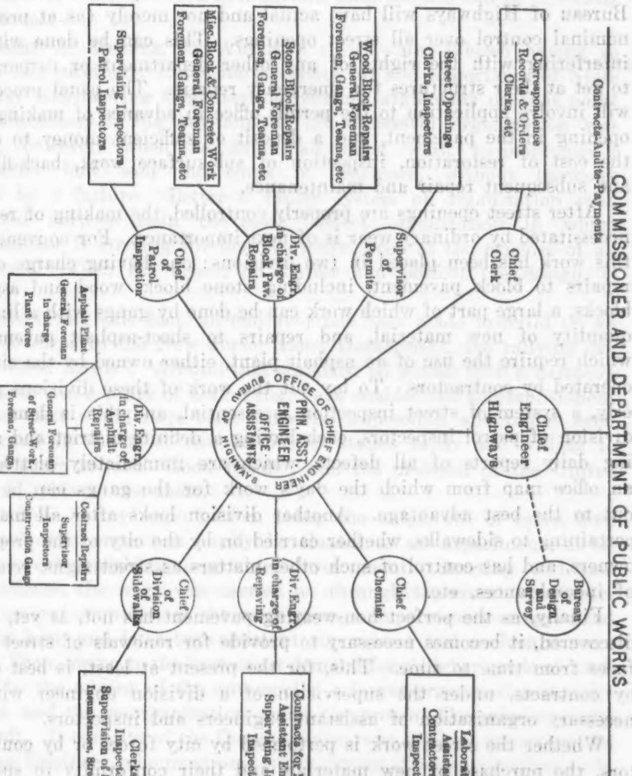

FIG. 2.

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With an organization such as has been outlined, and with the proviso of the enforcement of proper co-operation on the part of other departments and sub-surface corporations, and the co-ordination of their plans in general harmony for the greatest good of the greatest number, it is believed that street surface maintenance can be efficiently carried on. Mr. Durham.

ROBERT A. MEEKER, M. AM. SOC. C. E.—Mr. Whited having so clearly and succinctly stated the necessity for and importance of State Highway Engineering Organization, further arguments on that point are unnecessary. The key-note of all successful organization was struck, however, when he said "all must work harmoniously"; no matter how well the corps is organized, without harmony it will be a failure. Better a faulty system of organization with a united purpose than the most perfect organization in which each member is striving for his own glory, even at the expense of all the others. Then the result, no matter how perfect the design, is anarchy. Mr. Meeker.

Appreciating this fact, an endeavor has been made to build up an *esprit de corps* in the New Jersey Road Department, the rallying cry being "Good Roads". Around that standard all gather with a harmony of purpose and a unity of action that have enabled the accomplishment of much that would have been otherwise impossible, and to-day even the farmer in the most remote district stands ready and willing to pay his share toward "Good Roads", and the motorist of the United States delights to use them.

The New Jersey organization, being the outgrowth of more than 20 years' experience, can hardly be classed as an experiment; it is, more properly speaking, a gradual development. Starting with one commissioner, the work was carried on through the county engineers of the several counties; as the work grew, a supervisor was appointed to assist him, and later two assistant supervisors and a chemist were added. Then, owing to the increasing demand for a higher grade of road, caused by the advent of the automobile, a larger organization was demanded, and the re-organization under the present Commissioner was the result. It consists of the Commissioner, who is chief executive, the Highway Engineer, who must pass on the design of all engineering work, four Division Engineers, three of whom have each charge of a prescribed district and the fourth has charge of the bridges, a Chemist, who tests all road materials, twenty-one county engineers, who, though employees of the county, are also resident State engineers in their several counties, ten inspectors, and six foremen; the latter are to form the nucleus of a force that will be needed when the State highway system becomes an accomplished fact.

The foremen and inspectors report in writing weekly to the Division Engineers, who in turn report to the Highway Engineer and he

Mr. Meeker.

reports to the Commissioner, thus each has his alloted task and each his share of responsibility. The county engineers might properly be classed as brigade commanders having command of the work in their several counties and being subject only to general orders from headquarters as to standards that must be met.

The State highway system, just referred to, is also a natural outgrowth of the work, its aim being to unite the several county seats and to extend the main highways to the borders of the State. This being such a large proposition, it was deemed wise to establish a Highway Commission which should determine the general plan and location of the work. This Commission consists of the Governor, the President of the Senate, the Speaker of the House, the State Treasurer, and the Commissioner of Public Roads. This Commission has laid out a system of roads 1 500 miles in length, most of which are already improved; hence the State's principal task will be one of maintenance, which, owing to the fact that New Jersey lies between the two largest cities on the Atlantic seaboard, will always be a big one. Owing to the fact that many of our main highways have been laid out for from 100 to 200 years, their exact location is often hard to determine, hence it was necessary to add a right-of-way engineer to the force, and his services have proven so valuable that, in all probability, others will have to be added.

In closing, a word of tribute to the contractors should be added, for to them the engineers owe much; it is their practical skill that makes the designs and plans of the most perfect engineering organization possible.

Mr. Sargent.

PAUL D. SARGENT, M. AM. SOC. C. E.—The one argument that has been most effective in every State in bringing about the creation of a State highway department has been incompetency and inefficiency on the part of local officials in securing economical results in the expenditure of highway funds. Consequently, every State highway engineering organization, to prove its right to exist, must be developed along the lines of efficiency, competency, and honesty. Politics, in the ordinarily accepted sense of the term, can have no place in such an organization.

It seems to the speaker that the best results will be obtained and there will be more general satisfaction when the work is in charge of a commission of three members, provided they are men of sufficient breadth of view to understand their problem in its largest sense.

The commission should formulate the general policy to be adopted by the department, unless this has already been covered by law, and should pass on all large questions of a business nature.

Unless the commission is in daily session, which is rarely the case, it will be necessary for it to select an executive officer, who will usually be the chief engineer.

Besides engineering ability, this man should possess a good degree of tact, for meeting and dealing with the public, especially local officials. This is particularly desirable where highway improvements are paid for jointly by localities and the State, for this always means that the State directs the work, and, unless handled just right, local officials will feel that their authority is usurped. Other things being equal, tact in dealing with local situations should be a controlling factor in selecting subordinates all through the department. Mr. Sargent.

The chief engineer should have absolute control of the appointment of all subordinates, and, of course, they should report to him. The size of the organization will depend entirely on local conditions, that is to say, the quantity of work to be done in a given time and the area over which the work is distributed.

The following organization seems to the speaker to be about as small as can be expected to handle efficiently any considerable quantity of work: A division of surveys and plans; a division of design and construction; and a division of maintenance.

The division of surveys and plans will make all surveys and secure all preliminary data necessary for a proper design of contemplated improvements. This division should furnish plans of the road as it now is to the division of design and construction, where, after examining the locus, with plans, profiles, and sections in hand, changes of alignment and grade and details of construction will be determined.

The plans will then go back to the division of surveys and plans for completion, including preliminary computations of quantities, on which an estimate of cost will be prepared and bids will be invited.

At the same time the division of design and construction will prepare specifications, advertisements for bids, contracts, etc.

After the award is made and work is in progress, the supervision and inspection will be done by the division of design and construction.

It can be readily seen that in a small organization one man might handle both of these divisions, as their work is intimately correlated.

The division of maintenance, as the name implies, will look after this work especially, and it is suggested here, entirely on the supposition that both the other divisions are fully occupied with their particular problems.

This whole scheme is based on the supposition that there will be no division offices, as there are in several State organizations, but that all engineering work is done at department headquarters.

To complete the organization, there should be a division of tests and one of accounting. Tests should by all means be reported to the chief engineer through the division of design and construction.

When the chief engineer is the executive officer, the division of accounts should report to him, in order that he may be informed of the fiscal operations of the department.

Mr. Sargent. When a State highway department is in charge of a single commissioner, and a competent civil engineer is selected for the position, the scheme of organization herein outlined will be applicable by substituting in this discussion, the word "commissioner," for "chief engineer."

Mr. Crosby. W. W. CROSBY, M. AM. SOC. C. E.—In an efficient organization for producing physical results from men and materials, the responsibility for the results must be clear—if possible, individual—and the authority and resources in any case must be sufficient, not only to equal or balance the responsibility, but also, in order to be on the safe side and to avoid any arguments in the matter, to be appreciably in excess of the required duties and responsibilities.

In highway work, a division of the consideration is frequently made between Construction and Maintenance, and, perhaps, naturally, up to comparatively recent, if not actually the present, time, the bulk of the investigation and discussion in the United States has seemed to center under "Construction". It is beginning to be realized, however, that "Maintenance" is worthy of at least equal consideration, and that the proper organization of the maintenance forces is of prime importance.

The speaker, after long experience and wide observation of the experience of others, has had at least one conclusion clearly crystalized therefrom. That is, that, not only for the sake of securing prompt satisfaction in the results of the expenditures for labor and materials toward maintenance, but also for the sake of economy and efficiency in the long run, it is necessary to separate the maintenance work from the construction, and thus clarify the responsibility for the proper performance of each.

Naturally, there is no need for a State Commission or Commissioner for the Construction and another for the Maintenance, nor of two Chief Engineers on these lines; the segregation may properly begin immediately below the Chief Engineer, or it may not begin until the Division Engineer, or even the Resident Engineer, is reached. Local conditions and due regard for the rest of the whole organization of the department will decide this point. Ultimately, however, the separation of work and responsibilities must be reached.

The speaker has worked up through the matter from the beginning, of all construction work and no maintenance, to the point where the maintenance mileage far exceeded that under construction, and he has tried the various methods of placing the responsibility for the maintenance on the construction engineers, or maintenance engineers, and on others. He has had the maintenance work done by patrolmen, by gangs, by contract, etc.; but satisfaction has been approached only through such arrangements as made the maintenance work and responsibility for its proper performance the sole duty of the employees up

Mr. Crosby.

to a certain point where dependence could be had on that individual's appreciation of its proper relation to the importance of construction, and on whom the construction demands were not so great as to incite any neglect of the maintenance.

As has been previously stated by the speaker:*

"The importance and economy of prompt, efficient and sufficient maintenance cannot be overestimated in road administration, and this is especially true in the case of modern roads under present traffic conditions. More of the existing defects in roads today, whether such defects be those of appearance, of comfort, or of economy, are due more to weak points in maintenance than to deficiencies in construction or to any other cause.

"The requirements of maintenance work demand the careful performance of little things and the prompt attendance to such; persistence and continuity of action; good judgment and craftsmanship in actual work; and a high regard for economy and the old fact that 'many a little make a mickle'. Maintenance work contains far less of the spectacular than does construction, and thus some other stimulus is frequently needed by the workers in it that interest may not flag. Without some such incentive, the results are almost sure to be unsatisfactory."

Another matter to which the speaker wishes to call attention, in connection with the discussion under Organization, is that of the duties of resident engineers and inspectors.

The differentiation between these two cogs of the machine does not seem to be always as clear as it should be, and frequently, from lack of such clearness, duties seem to be extravagantly imposed on one or to be loaded on the other so as to cause a failure in the operation.

A resident engineer is generally supposed to be a man of some experience and skill in the work under his charge, and who, on account of such expertness, is delegated from above certain (limited) authority for making decisions and for directing the details of his work so as to facilitate the progress of the latter and relieve his superiors from the necessity of considering and deciding minor details when once general directions have been issued in one way or another by them. It should be noted that ability, experience, firmness, good judgment, and tact are primary requisites for a satisfactory resident engineer.

An inspector, on the other hand, is merely the eye of his superior, to whom he reports, focussed on the particular work on which he is stationed for the purpose of observing or inspecting its performance. He is instructed, by means of copies of the clauses of the specifications that apply to that work under his observation, or by other means, as to what is required for satisfaction of the contract in such work, and, in case of any apparent evasion or violation of the provisions of his instructions, is or should be directed to call the attention, first, of

* Report to the State Roads Commission of Maryland, 1912, p. 101.

Mr. Crosby. the contractor and, secondly, of his nearest superior with authority, to the matter.

An inspector, as such, has or should have no authority over the work delegated to him, and his responsibility is, or should be, accordingly limited to reporting promptly on the facts as he notes them. To delegate authority to him makes him no longer an inspector but rather a resident engineer of more or less limited field, as the case may be.

It is not the speaker's intention to provoke discussion of the etymology of the two expressions, or to quarrel with their various uses, but rather, by the foregoing, to illustrate differences of meaning that may be assigned to each for the main purpose of clarifying and, as far as possible, lending force to the following remarks.

The speaker (and doubtless those to whom these remarks are addressed) has continually heard references to difficulties supposed to be omnipresent between contractors and engineers, and much time on many occasions has been occupied in discussing such differences or sources of friction and means for their reduction or relief.

From a long experience, in capacities ranging upward from inspector to chief engineer, the speaker has become convinced that the responsibility for such friction rests largely in many cases with the chief engineer or other party responsible for the organization of the engineering forces for highway work.

In connection with the actual organization of the engineering forces, consideration of the specifications for the work itself must not be neglected, for, in part at least, they generally affect the operation of the machine, or express in words what is graphically or mechanically shown by the form of the organization itself.

In a lecture at Columbia University a year ago, the speaker made the following statements which it seems to him are pertinent for repetition here:

"With a proper organization of an engineering department, including the definitely understood delegation of authority, the full expression of the work to be required by plans and specifications, the location of the responsibility on the proper party (the chief engineer) and this recognition of their obligations on the part of the contractors, ninety-nine per cent. of the present causes of friction between the latter and the engineers would be removed and in place thereof would be substituted the cordial assistance of the engineers to the contractors.

"To hitch up an experienced contractor's foreman, bent only on making money for his employer, on one side of the pole with a young, ambitious but green, inspector (perhaps the only one obtainable) on the other side, and then to turn this team loose and to expect them to keep steadily along the road toward success would be folly, yet it is often attempted, for valuable inspectors do not often remain as such as long as do foremen; but if the foreman realizes that the inspector

is to call for help the moment he feels he is losing his footing and that the call will be answered by an experienced man fully capable of handling the situation, there is far less likely to be the need for such a summons. Few competent engineers and contractors cannot get along together peaceably and advantageously. The difficulties seem to come largely from mismatching, by improper delegation of authority, and from lack of clearness and definiteness in describing the work and materials to be furnished. Mr. Crosby.

"The speaker knows from experience the difficulties of the young, enthusiastic, impatient resident in preserving a placid equanimity day after day when constant evidence of a desire on the part of a contractor to evade the specifications, to substitute 'something just as good' for the materials and work he has agreed to deliver, is offered. On the other hand, he sympathizes with the contractor who is constantly being nagged by an inspector over minor deficiencies, or when the inspector tries to 'run the job himself'. Perhaps it is human nature for most contractors to try out an inspector to see what shortcomings he will quietly submit to, and, having found out, to his advantage or loss as may be the case, thereafter to gauge his work accordingly. Perhaps it is natural for most contractors, after competitive bidding on public work, to attempt to substitute at every possible opportunity 'something just as good' for the article specified. But the speaker believes that much improvement in the relations between contractors and engineers would be had were the organization and specifications, as before referred to, made most explicit and then the contractors to regard their carrying out to the letter as much an obligation on themselves as the completion of the work."

Therefore, in framing an engineering organization for highway work, and in drawing the specifications for the work under it, the speaker believes that full consideration should be had, previous to arranging for the resident engineers and the inspectors, of the possibilities of securing competent "residents" for the salaries fixed to be paid; of the possibilities of securing alert and satisfactory "inspectors" for the salaries fixed for these positions; of the competency and character of the contractors likely to carry on the actual work, and of other conditions likely to affect the decision, and then to design the machine in all its details so as best to fit all the conditions.

It is scandalous for an engineer to design a machine requiring steel in all its parts and then, on that design, to build it of wood because wood happens to be the available material. It will be disastrous to allow the use of wood for a cog when steel is known to be needed, and every cog in the machine, even the least important, should be recognized as needing in its character an ample "factor of safety".

One word further: Such a machine should not be designed or built too delicately or too lightly. The larger and more complicated the machine, the more necessity there is for sufficient mass in the machine parts themselves to take up the shocks and vibrations, from both without and within, incident to its operation. Mass is necessary for

Mr. Crosby. momentum, and a sufficient momentum is desirable in this case for the purpose of continuing action in spite of the inevitable obstructions in regular and efficient operation.

Mr. Lewis. NELSON P. LEWIS, M. AM. SOC. C. E. (by letter).—It being admitted that an engineering organization is required to deal with highway work, it follows that the details of such an organization should be determined by engineers. The time has passed when engineers should be expected to carry out municipal work of any kind when they are told, as they are told, and because they are told. Municipal engineers must assume the responsibility of determining, not only the kind of organization through which this control is exercised, but the general policies of a city with respect to such work. Every city needs a definite plan and programme. The need of a comprehensive plan for a street system is now generally recognized, but it is not as generally recognized that this should be the work of the engineer. A paving programme is just as essential. The policy of most American cities has been to leave the kind of pavement to be laid on any street to be determined by the abutting property owners, on the ground that, as they are to pay the bills, they are entitled to say what kind of pavement they should have. Such a plan cannot result in a well-paved city. By a well-paved city is meant a consistently paved city, that is, a system of pavements in which the surface on each street is determined by the grades, the traffic, the development of the abutting property, and its ability to pay. If grades permit, it is desirable to have the same kind of pavement on a continuous thoroughfare, and it is very desirable that the driver of a team should be able to follow a pavement on which horses can travel under any weather conditions from any part of a city to any other part.

There should also be a definite plan for the location of sub-surface structures. The building of pipe subways has frequently been urged as the only means by which an orderly arrangement of underground structures can be secured and the constant opening of the street surface prevented. When a new street is being built or an old street is being widened and entirely reconstructed, the placing of pipes and conduits in subways may be practicable, but, unless all these pipes are new, and unless the greatest care is exercised to prevent leakage, especially of gas, the risk of carrying out such a plan is considerable. Paris no longer permits electric wires to be placed in the sewers, where the water and gas mains are accommodated, but separate electrical conduits are usually placed under the sidewalk. This, again, is a subject for careful engineering investigation, and is worthy of assignment to a place in an engineering organization dealing with highway work.

In the opening discussion, the city highway bureau was aptly described as the principal show-case of the city government, the pave-

ments representing the goods in the window. It follows that there is no department of municipal work where inefficiency is so sure to result in criticism as in the highway bureau, but the organization to deal with this branch of municipal work should not be limited to the mere laying and maintenance of pavements and their cleansing; it should extend to the planning of the lines and grades of the streets, the selection of the type of pavement for each street, a rational method of financing the cost of a street improvement, and also the proper placing and care of the underground structures, which do not appear in the show-window, but which, nevertheless, are an important adjunct to every street. Mr. Lewis.

JOHN C. TRAUTWINE, JR., ASSOC. AM. SOC. C. E.—Mr. Durham's discussion calls attention to what may be called two systems of stratification, in the matter of highway organization: (1) The vertical system, in which each of the several geographical divisions of a large city constitutes, in effect, a little sovereign state, with its own highway organization, independent of those of the other divisions; and (2) the horizontal system, in which a single highway organization presides over the highways of the entire city, including all its sub-divisions; as Pennsylvania's blanket of "new red shale" overlies the upturned edges of many much older formations. Mr. Trautwine.

Now, although Mr. Durham expressed his preference for the second, or horizontal system, the vertical system appears to obtain in New York City, where the Borough of Manhattan, the Borough of Queens, the Borough of the Bronx, and the Borough of "What-not," each constitutes apparently a little highway principality, with sovereign powers over its own affairs, and permitted to act (if it chooses) in imperial disregard of the arrangements and interests of the others; so that, conceivably, two adjoining boroughs, in an outburst of local patriotism, might so revise the alignments and grades of their highways that those of one borough should have no physical connection with those of the other.

Now, to paraphrase Sterne (who has been quoted in this discussion) the speaker submits that "we do these things better in Philadelphia." Mr. Connell will correct the speaker if he is wrong; but he is under the impression that Mr. Connell reigns supreme over the highway organization of that city, and that there are no highway chiefs of Manayunk, or of Chestnut Hill, or of Frankford, or of Southwark, empowered to regulate highway matters, in their several districts, without consulting him.

Philadelphia has the horizontal system of stratification; and this is in accord with the modern tendency to centralization, with its necessarily higher efficiency.

The programme for these meetings mentions State and Municipal Highway Organizations. The speaker ventures the prediction that,

Mr. Trautwine. within 50 years, we shall have a National highway organization, administering the streets and highways of the entire nation, and of its cities, from whatever is then the national capital; and that, within a century, we shall have a World highway organization, similarly administering, from a central office, the streets and highways of civilization —with universal benefit—on a scale now undreamed of.

The same will be true, of course, of matters like water supply and sewerage, now in general conducted (on the vertical system of stratification) by each city for itself; although Massachusetts (in the advance, as usual) has set a notable example in the formation of its Metropolitan Water and Sewerage Board, which regulates these matters for Boston and its neighboring communities, each of which formerly looked out for its own interests, mostly without reference to those of all the others.

Of course, it is not to be expected that the suggestion of a national and a world highway organization will be looked on with favor by incumbents of municipal offices which, under such organization, of course, would have to accept subordination in exchange for their present sovereignty, unless indeed from those who felt pretty sure of being called to the national or world chiefship.

Mr. Goldsmith. WILLIAM GOLDSMITH, ASSOC. M. AM. SOC. C. E.—There is one detail in Mr. Connell's discussion which has proved of vital importance to the organization of the Highway Bureau in the Borough of Manhattan. This relates to patrol inspection. In past years civic organizations and private citizens have complained repeatedly about the poor condition of the pavements in Manhattan Borough, and particularly with reference to the length of time elapsing between the opening of a pavement and its final restoration. These complaints culminated in an investigation by the Commissioners of Accounts during 1906 and 1907. Their report showed:

First.—That there was no system which made it possible to follow up cuts made in pavements so that they could be quickly restored.

Second.—That the responsibility was not fixed, and the shifting of blame from one to another was not only possible, but was the case whenever difficulties arose.

In 1910, under Borough President McAneny, steps were immediately taken to remedy these conditions. The Bureau of Municipal Research, working in conjunction with the Bureau of Highways, inaugurated an inspection system, which, during the last 2 years, has worked admirably. Although this is a detail in highway organization, it is vital to success, and is worth while noting.

The city is divided into districts, for each of which there is a patrol inspector. His territory covers about 10 miles of pavement, there being about 450 miles of paved streets in Manhattan. It is the

inspector's duty to report at least once a week regarding the general condition of the pavements, and to report promptly on all street openings, building operations, and encroachments for which permits are issued. Mr. Goldsmith.

In order to give a clear idea of the inspection system referred to, take, for example, a plumber's street opening. The plumber must be licensed (pass an examination as to fitness, before the Board of Plumbing Examiners), and must execute a $1 000 bond with the city for the proper performance of his work. He comes to the Permit Division, which is under the direction of the Bureau of Highways. A technical man, called the Supervisor of Permits, is in charge of this Division, and keeps in direct touch with contemplated pavement work, sewers, or other constructions; sees that sub-surface corporations do their work in streets previous to resurfacing; that all departments are notified of contemplated constructions; and approves every permit before it is issued. The plumber makes a cash deposit covering the probable cost of pavement restoration. This payment includes an inspection and overhead charge (which amounts to approximately 50 cents per sq. yd.). He must also make a cash deposit of an equal amount, which is retained for 6 months, to insure the city against any settlement of the pavement, due to improper back-fill, or other poor workmanship.

As soon as a permit is issued, triplicate inspection forms are made out on a typewriter in the Permit Division. These forms provide space to give the inspector in the field all the information necessary for a proper inspection of the work. These forms are called: (1) a "Preliminary"; (2) a "Tickler"; and (3) a "Final".

The "preliminary" report is given to the inspector, whose duty it is to see that the conditions of the permit are complied with, to report thereon what work was done, and, by actual dimensions and sketch, show just how much pavement was disturbed, and the quantity to be restored and its exact location.

The "tickler" and "final" forms are filed in a cabinet provided for that purpose in the main office. When the "preliminary" report is received from the inspector, the "final" is sent to the pavement contractor, who honors it as an order from the city to restore the pavement. The contractor hands this "final" to his foreman, who takes it out on the work and makes the necessary repairs in amount and location as shown.

With these repair gangs there is another set of inspectors, called maintenance men, who are entirely independent of the patrol inspectors. The foreman of the repair gang hands this "final" report form to the maintenance inspector assigned to his work, and it is this inspector's duty to see that the pavement, of the exact amount and location shown on the inspection form, is repaired according to specifications. He then records the actual amount repaired and signs this "final" form in

Mr. Goldsmith.

a space provided for that purpose. These maintenance inspectors are notified by a chief inspector, about 18 hours in advance, just where their work will be on the following day.

After the day's work is completed, the inspector fills out and signs the "final" reports, and mails them to the main office. In this way, an independent check is obtained, and collusion is almost impossible between the inspectors and contractors as to the area of pavement to be restored. The paving company then presents its bill for this restoration work to the city, and a clerk in the office, uses the "final" reports as a basis on which to approve the bills.

During 1913 an experiment on patrol inspection was tried, the object being to reduce the number of patrol inspectors and put them where the work was urgent. The territory north of 59th Street and west of Fifth Avenue, in Manhattan Borough, was patroled by one inspector in an automobile. Previously, this section had been patroled by twelve men. The difference in cost was:

	12 Inspectors at. .$1 200 =	$14 400
as compared with:		
	1 Inspector at. . 1 800	
	1 Chauffeur at. . 1 200	
	Maintenance and depreciation of automobile 1 000	
	Total	4 000
Yearly difference, or saving....................		$10 400

This covered a mileage of approximately 132 miles and gave a patrol inspection cost per annum of $30 + per mile, as compared with $109 + per mile by the old system, showing a saving of $79 per mile per annum, or a total of $10 400 per annum.

Although this scheme worked out well in the district where it was used, it probably would not be as effective in heavy traffic sections, where the territory covered would be limited on account of congestion and where the speed of the automobile would not be an advantage.

The inspection and follow-up system referred to, which includes a number of patrol inspectors and an independent number of maintenance inspectors, has the following advantages:

(1) It places individual responsibility on the inspector on the work, as the reports must be signed by him;

(2) It makes the shifting of responsibility impossible;

(3) Possible collusion between inspector and contractor is out of the question;

(4) Experience shows that better results are obtained.

The fourth statement is easily verified by a comparison between the present state of the streets in the Borough of Manhattan and their condition under the old system. Mr. Goldsmith.

In a highway organization, the details of the inspection system are vital to a successful administration, because the success or failure of highway work depends on the field inspection. The speaker believes that the system outlined is feasible, covers all necessary requirements, and, if properly supervised, will get rid of many of the difficulties with which a city highway engineer must contend.

GEORGE A. RICKER, M. AM. SOC. C. E.—The speaker is glad of the opportunity to say a few words about the reorganization of the New York State Highway Department, and the methods that were followed in selecting the Division Engineers. When Commissioner Carlisle and the speaker undertook this reorganization there were only three regular Division Engineers on the work, and, as the new Highway Law commanded the redivision of the State into nine divisions, there were six vacancies to fill. It was somewhat troublesome to think of how to secure competent engineers for these divisions who, at the same time, would be good executives, for it must be borne in mind that these divisions are very large, each containing a mileage of roads as great as the whole of Massachusetts. Mr. Ricker.

It was thought to be practicable to secure by civil service examination and appointment men who had the necessary personality, men with executive force and the ability to command. By permission of the State Civil Service Commission a plan of co-operation with the Highway Department was established in the preparation and conduct of an examination, which consisted of three parts. First, an oral test (and there is point in its being placed first); second, two theses, one on the subject, "Essentials of Highway Construction," and the other on any large work in which the applicants had had experience; the third section consisted of a sworn statement of each man's experience.

There were about 160 applicants. One by one the men were called into the office of the Civil Service Commission on a given day, and two members of the Board of Consulting Engineers and the speaker sat with the Commission. An attempt was made to put the candidates at ease by the attitude of the examiners, and by asking them unimportant questions until any nervousness which they felt was overcome. After a few minutes' conversation the leading questions, calculated to bring out the characteristics of the candidate, were put. It was assumed that the candidates were ready with technical matter, but technical questions were avoided, as the examiners were quite sure that the men were well equipped along that line, and if not, that would be found out in the later tests.

The object of placing the oral test first, it will readily be seen, was because men of executive force and strong personality were needed.

Mr. Ricker. They might have all the technical knowledge in the world, all the education and experience necessary, but, if they could not control men, they would not make good Division Engineers. By this test it was possible to exclude from the examinations that would follow men who were lacking in necessary personality to make good executives. Some fifty or sixty men were selected, whom it was thought were very desirable, and they were instructed to enter the other tests. It should have been stated, by the way, that as each man left the room the examining board agreed on his mark before the next man was called.

The entire board read and criticized the theses, and, as a result of the complete examination, the Civil Service Commission presented a list from which six men were selected from among the first seven men: one from the Board of Water Supply, three from the Barge Canal organization, one from the Highway Department, and one who had had six years' experience in the Panama Canal work. These six men, with the three from the old organization, constitute the Division Engineers of the reorganized Department. All these men are members of this Society, and all are also graduates of recognized engineering schools, the several institutions represented being the Massachusetts Institute of Technology, Cornell, Rensselaer, Polytechnic, and Marietta College (Ohio).

Following this start in the reorganization, each of the Division Engineers has been supplied with an Assistant Division Engineer, the latter from promotions in the Department. An examination was held, following a plan similar to that outlined for the Division Engineers, and twelve men from the first thirteen were appointed, some divisions getting two men on account of their size and the magnitude of the work. These Assistant Division Engineers, technically known as Resident Engineers, were taken, with one exception, from the top of the list down. The one man passed, as in the case of the Division Engineers' list, was not passed on account of any political reasons. Politics had absolutely nothing to do with the selection of any of these men. They were chosen and appointed before any one of them knew that they were being considered, so that nobody had any chance to "see the Commissioner" or the speaker regarding them, and they had no opportunity to send anybody on their behalf if they had wanted to, and, as far as the speaker knows, none of them ever thought of doing such a thing. Thus it is seen that the reorganization is purely on a merit basis.

Arrangements are now being made for the selection of Assistant Engineers to place in charge of counties. The Division Engineers have recently met and discussed the list of available men presented by the recent promotion examination, and have agreed among themselves as to the disposition of these assistants throughout the State by counties.

Mr. Ricker.

An attempt is being made to establish something approaching a military organization, and the Division Engineer in complete charge of his division will be supported and sustained. Politics will have absolutely nothing to do with the work if it can be prevented, and it is thought that it can. Commissioner Carlisle, although not an engineer, has engineering sense, and in a marked degree possesses executive and administrative ability.

In January, 1914, there was established a system of weekly reports, which are really time sheets, so that every man in the Department reports to the Bureau head and Bureau heads send to the Department head a record of what every man is doing every day. Thus it is known where the men are, and what they are doing, and in that way one can readily keep track of the force.

Mr. Washington.

WILLIAM DE H. WASHINGTON, ASSOC. M. AM. SOC. C. E.—In connection with the discussion of the subject of highway engineering organizations, the members of the Society are referred to a report to the Hon. John N. Carlisle, Commissioner of Highways of the State of New York, by the Board of Consulting Engineers, consisting of Harold Parker and George C. Diehl, Members, Am. Soc. C. E., and the speaker. In addition to covering specifications for various types of roads and pavements and the materials to be used therein, the report considers the subject of efficient organization for the State Highway Department of New York, and gives in detail the work to be covered by such an organization, the duties of the various engineers included in the organization, and charts covering the inter-relationship of its component parts. The report is on file in the Library of the Society.

Mr. Whited.

WILLIS WHITED, M. AM. SOC. C. E. (by letter).—In reply to the very able and interesting discussion which has been elicited on this subject, the writer would call attention to the following points:

Mr. James includes Pennsylvania among the States in which road taxes are paid in labor. This was abolished by an Act of Assembly in 1911, and all road taxes are now paid in money.

Much is said on the subject of the construction and maintenance of improved roads, that is, roads covered with some kind of paving. More than 90% of the roads in the United States are not improved, in the sense mentioned, and probably will not be for many years to come. The most important problem that confronts the State highway administration is to make these roads as good as possible at a minimum expense, and generally with a view to more permanent improvement at some future time. If an engineer has work to do that is worth \$10 000, and has \$10 000 with which to do it, the proposition is comparatively simple, but if he has \$10 000 worth of work with only

Mr. Whited. $1 000 with which to do it, a very high grade of engineering ability is called for.

Engineering is a union of common sense and science. It deals with the forces of Nature; but by far the most difficult force with which the engineer has to deal is human nature, and ability to deal with human nature can hardly be acquired in any college course. It requires genius, to begin with, and proper training, which includes a good knowledge of the fundamental principles, including moral principles, and the ability to apply them effectively.

Mr. Durham's remarks bring out strongly the fact that although the organization in Paris is almost ideal, the results are open to criticism; whereas, in London, notwithstanding the fact that the organization contains obvious defects, the results are excellent; and in Berlin where everything is under the control of a Government which has a genius for organization, the results are variable. These facts point strongly to the indication that administration is a personal matter, rather than a question of form of organization, and, to produce the desired result, it must be sufficiently elastic to permit of all men being assigned to the duties for which they are best adapted.

The French are very systematic and competent, technically, but inferior to the English in executive ability. The Germans are very skillful and industrious, but are so accustomed to military rule that difficulty is apt to develop in case of an emergency requiring special methods.

The writer is often inclined to deplore the position of many in the Profession who advocate the placing of engineers at the head of all work that is even partly of an engineering nature. The combination of a high grade of technical skill and knowledge with the sure and accurate judgment of men that is required in any high administrative position, together with the proper understanding of the many legal, commercial, and political questions involved in State highway administration, is so rare as hardly to be looked for on this earth; and, for a man at the head of such an organization to be notably deficient in any of the latter requirements is almost certain to wreck the department; whereas, a man possessed of these qualifications, as well as the self-control and tact required for such a position, can usually make a success of it, even though he is equipped with but a moderate amount of technical knowledge. A man possessed of the latter, with a sufficiently elastic organization, will have the power and the disposition to avail himself of any special knowledge or skill possessed by any member of his staff, whatever his title may be, in the solution of any problem that may present itself. Every man has his weaknesses and deficiencies, and it is of the utmost importance that the "Chief" see that none of them is allowed to interfere with the smooth running of the organization.

The writer would suggest that the most important qualifications of

the head of any department of public works are tact, common sense, self-control, and the ability to deal with men, both personally and commercially, and a knowledge of his legal rights and duties, as well as those of the traveling public, the public service corporations, and the abutting property owners. The more knowledge of engineering technology he has the better, but the questions which arise in strictly administrative work are generally questions which must be settled on the spot, while those of a technical nature can usually be referred to some expert in paving, bridge construction, tests of materials, etc., who should be included in the staff of the department. Mr. Whited.

A word as to the politician: The general idea is that the politician is a professional criminal, whereas, the fact is that politicians of the higher grade are fully as honest as men in most other lines of business, and are not only possessed of tact and common sense, with a thorough knowledge of the world, but are also industrious as a rule.

As to the statement by Mr. Bennett, to the effect that a prominent engineer has said that there were few engineering problems to be faced in highway work, the writer would say that a good definition of an engineer is, a man who can do with one dollar what any one can do with two; but, considering the many millions of dollars spent annually on road construction and maintenance, and the importance of the interests involved, there is ample scope for the exercise of the best grade of engineering ability.

It is doubtful whether anybody has reached, or ever will reach, a final settlement of the conflict between the inspector and the contractor's foreman, at least in the present state of human nature. The strictly competent inspector does not exist, and never will, and the same may be said of every one connected with the work. The only thing to do is to fight out each individual question as it arises. The writer will suggest, however, that a frequent cause of the differences between the contractor's foreman and the inspector of small and limited experience, is that the foreman, or some of his workmen, with or without the knowledge of his superiors, will in some way get the better of the inspector when he is off his guard, and the inspector finds it out when it is too late. The anger, suspicion, and hostility which takes possession of the inspector need not be explained to any one understanding human nature. It is easy to deceive anybody, but it is almost certain that it will be found out eventually.

Regarding efficiency boards, the writer would say that the efficiency expert has his place in all public or private work, but his outgivings can seldom be taken as final, because so much depends on the personality of the officials and employees, both high and low.

WILLIAM H. CONNELL, ASSOC. M. AM. SOC. C. E. (by letter).—The discussions on "Municipal Highway Engineering Organizations" would seem to indicate that there is a general tendency to raise the Mr. Connell.

Mr. Connell. standard of organizations controlling this important branch of engineering work. The exceptions taken to the organization advocated by the writer are matters of minor detail, and in some cases are due to a misunderstanding. A few of those who discuss the subject are apparently under the impression that the writer advocated a standard highway organization. This, of course, is not the case, as such an organization would not be possible on account of the varying sizes of the municipalities; but an attempt was made to outline the general principles which could be followed in forming an organization suitable for municipalities of various sizes. Some seem to dwell at length on coping with existing conditions. That, of course, is necessary, but is simply routine; it would not do, under any conditions, to plan an ideal organization which could not be operated successfully under different laws, ordinances, and existing conditions, over which no control could be exercised; and, therefore, the first thing to do is to plan the work in such a way that it can be carried on successfully and economically under existing conditions, and try to have these conditions improved so that the ideal organization can be put into operation in the course of time.

One of the questions of detail, on which there seems to be some difference of opinion, is whether there should be a separate organization for maintenance. It would seem to be logical that the organization to take charge of the maintenance should be the one under which the pavements were constructed, as the men in that department understand all the conditions in connection with that work—some of which might at a later date be factors in the maintenance problem. Having a single organization also does away with the shifting of responsibility. The maintenance is just as important a part of the road problem as the construction, and no pavement should be laid without taking into consideration, among other things, its probable life and the probable maintenance charges during its life.

Another very important consideration, in having one organization in charge of both branches of the work, is that it tends to build up a better personnel and makes the men more generally useful. In other words, an important part of the education of the highway engineer is to understand and realize the importance of the maintenance problem. A good many of them do not realize the importance of this problem because, in many cases, separate divisions are in charge of the construction and maintenance, and the men who have nothing to do with maintenance, as a rule, devote very little time and study to the subject. There is no doubt that more study would be given to the general highway problem if all the work was under one organization; the engineers who constructed the pavements would know that they were to be held responsible for the maintenance, and would naturally give more time and thought to this phase of the work. Aside from this fact, it would

Mr. Connell.

require a smaller organization if combined than if separate, as each division—no matter how well organized—tends to a duplication of work, to a certain degree.

One member advocates the general scheme of organization, but points out that the weak point of the system is in the question of personnel, and further states that in the United States there is no trained class of public works officials or Government engineers, organized like the French corps for bridges and highways. In view of this, he states that, in the light of possible conditions, it would seem to be better policy to concentrate along all classes of work, organizing the bureau in sections for doing specific things, in charge of the man best qualified to do them. This, of course, is a proposed scheme to cope with existing conditions, which is just what should be avoided. Engineers are often too apt to accept conditions as they are, instead of trying to improve them, and if they do not strive to form an organization that will stand and that is not dependent on any one man suited to fill a particular position, the organization can never be a stable one. A highway organization should not be dependent on any individual, but should be designed so that it would be constantly training men to fill the vacancies of those passing out, and it would appear to be better policy to endeavor to change the conditions that prevent formulating an organization of this kind, than to become reconciled to existing conditions.

A very important consideration in connection with the highway organization is the question of the *esprit de corps*. Mr. Meeker states that the key-note of all successful organizations is that all must work harmoniously. No matter how well the corps is organized, without harmony it will be a failure. This is a very important matter, and cannot be given too much attention as a factor in the control of a good working organization for this or work of any other class.

There was some discussion regarding the method of handling the patrol inspection. The writer advocated that the street-cleaning force should be combined with the general highway organization, and that the maintenance patrol inspection of the pavements should be handled by the street-cleaning inspectors or the foremen. The advisability of this method was questioned. This was advocated in order to increase the usefulness of the men in charge of the inspection work, and thus reduce the cost of the patrol inspection by utilizing men already on the work, such as the street-cleaning inspection force, instead of employing additional men to cover the ground every day. There is no reason why the police could not also be utilized for work of this kind, and when our municipalities are further advanced, and more on a business basis, there is no doubt that they will be more generally used in connection with matters of this kind than at the present day.

Mr. Connell.

If some general principles were formulated relative, not only to the character of the organization, but the details of conducting the work, resulting from discussions of this kind, they would carry a certain weight and would be a great help to engineers throughout the country in having the laws, regulations, and ordinances changed to make it possible to plan an organization in accordance with what is conceded to be the best practice by the authorities on this subject. In other words, it is always easier to put into effect anything that is the combined opinion of the engineers of the country at large than it is to accomplish it by individual effort.

(2) FACTORS LIMITING THE SELECTION OF MATERIALS AND OF METHODS IN HIGHWAY CONSTRUCTION.

By Messrs. P. E. Green, E. H. Thomes, S. Whinery, W. W. Crosby, Mark Brooke, George W. Tillson, Robert A. Meeker, R. E. Beaty, William M. Kinney, William Goldsmith, George A. Ricker, D. B. Goodsell, Henry W. Durham, William de H. Washington, and P. E. Green.

P. E. Green, M. Am. Soc. C. E. (by letter).—One of the rules adopted for this discussion is that the matter to be presented must be new. It is a very proper rule, but it should be realized that it is a very difficult one to live up to. As far as new matter is concerned, it is hard to conceive of a phase of this subject which has not been touched upon or investigated more or less thoroughly, except possibly from the laboratory end. It would appear necessary, therefore, to confine oneself to an academic discussion of the matter or to present individual experiences showing some variation from expected results. In order to bring out a broad line of discussion, it might be well, however, to systematize the factors entering into the problem and call attention to some phases which might be investigated a little more than has been done in the past, with the hope that some important new points may be presented. In opening this subject the writer will confine himself to a general outline of the problem, leaving the details to be discussed by those who follow. Mr. Green.

The subject as assigned may be logically divided into two interlocking general sections, namely: "The Selection of Materials", and "Methods of Construction". Each of these main headings may be further divided into numerous and important parts. All these parts should be taken into consideration by any public authority, such as a city, county, or State, when it is proposed to enter on a general scheme of highway improvement such as is so common at this time. In this way it is not difficult to make a logical selection of the various materials and methods involved in the construction of a system of streets or highways; and a scheme or plan of future construction may be laid out. This does not mean that only one material shall necessarily be selected for any one street or highway. It simply means that the class or type of material shall be selected, and that choice must ultimately be made between several more or less suitable products.

A diagrammatic outline of such a general scheme covering the subject of this discussion is herewith submitted. It is not claimed that it is particularly novel or complete, but is presented for discussion and improvement. Although intended primarily for country highways, it may be used with equal facility for city streets.

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DIAGRAMMATIC OUTLINE OF "FACTORS" IN ROAD WORK.

- **Highways under Consideration.**
  - I.—Selection of Materials.
    - 1.—Character of Road (City or Country).
      - Trunk.
        - Through traffic road—commercial or passenger, or both.
        - Commercial road, but not necessarily through traffic.
      - Secondary.
      - Local.
        - Connecting important centers to trunk lines.
    - 2.—Traffic.
      - Much light fast passenger; or
      - Heavy hauling; or
      - Only local hauling.
      - Roadway widths required.
    - 3.—Climate.
      - Northern.
      - Intermediate.
      - Southern.
    - 4.—Character of Supporting Country.
      - Property values and benefit to be derived.
      - Vicinity of large cities.
      - Topography—drainage.
      - Character of business using road—esthetic considerations.
    - 5.—Materials.
      - Class.
        - Superior.
        - Inferior.
          - Physical conditions governing, such as wear, crown, dust, sanitation, etc., etc.
      - Cost—maintenance charges.
      - Character and appearance.
      - Ease of handling materials.
      - Relation of material to roadway width.
  - II.—Methods of Construction.
    - 1.—Laws and legal organizations governing.
    - 2.—Location of railways with respect to improvements—railway rates.
    - 3.—Existing good roads as affecting haul—wagon or truck.
    - 4.—Availability of labor.

The scientific selection of the material to be used in the construction of a highway depends on all the items named in the diagram. The diagram itself looks somewhat complicated, but its use is very simple. Before commencing any large scheme of highway improvement, the outline may be studied with reference to materials, methods, etc., and simple tables prepared. Too often some essentials are not considered seriously or are forgotten, and for this reason much money, time, and labor are wasted. From time to time, somewhat similar schemes have been advanced, but frequently they deal with only one essential, such

as traffic, to the exclusion of other important elements. For example, in addition to traffic, the character of a city highway should depend on the layout of the city, its topography, population, business, education, and wealth; and, further, the location of the city as regards the availability of standard materials, such as brick, wood block, crushed stone, gravel, sand, tar, asphalt, etc., etc., is vitally important. Mr. Green.

Considering the problem of a country highway system, many entirely different factors enter into the selection of the material, such as vicinity of cities, large or small; general population and wealth of the surrounding country; whether or not the highway is to be a trunk line for through traffic, and if so, whether such traffic is light, high-speed traffic, or heavy, low-speed, commercial traffic.

If the highway is a secondary one, as regards the through traffic, it still may be a trunk line as regards its commercial traffic, such as a road leading from a trucking district to any one of our large cities. Such a highway may be comparatively a short one, yet the character of its traffic actually makes it a trunk line.

The least important class of country roads is generally referred to as "local roads". These are exactly what the name signifies, and whether or not they are to be improved depends on the wealth and education of the surrounding country and its inhabitants. Such roads are generally paid for only by local taxation.

The effect of "climate" is more important than is often considered. Some pavements, such as the various bituminous pavements, concrete, etc., are very much affected by the temperature. Thus a surface mixture of asphalt suitable for Chicago is not suitable for New Orleans; and concrete as a wearing surface should do better in Houston than Buffalo.

Three zones may be recognized: Northern, embracing the New England States, New York, Pennsylvania, Ohio, Illinois, etc.; the Intermediate, such as Virginia, Kentucky, Tennessee, Arkansas, etc.; and Southern, such as Alabama, Mississippi, and Texas, which have little winter. In the Northern States, the selection must be of materials not affected by extremes of temperature, freezing and thawing. For the Intermediate, it is not necessary for the materials to withstand great variations in temperature. For the Southern, materials which might utterly fail in the North because of inability to withstand extreme cold or great variations in temperature, may be very satisfactory. Furthermore, the exceedingly heavy and continuous rains following long hot, dry spells in the southern sections must be taken into account.

A great deal of money might often be saved by considering carefully the various classes of paving material, which may be suitable for the same purposes and yet may vary widely in cost in the different parts of the country. Thus, wood block pavement may be a very reason-

Mr. Green. able one for the business streets of small cities in regions close to the sources of supply, but for other cities, similar in size, but not so situated, it would be a grave mistake to use it, because of its prohibitive cost. This same wood block pavement has a characteristic in common with sheet-asphalt: it must receive a certain minimum traffic or it may fail. On a wide roadway, having only a traffic along the center, the blocks outside this used strip will curl up under the sun and swell and "blow up" if a rainy fall follows a hot, dry summer.

Further, such a material as trap rock, though making good macadam highways in New York, probably better, as far as wear is concerned, than any of the limestone used in the West or South, would be impossible for these last locations. Yet the writer has seen specifications, for roads hundreds of miles away from granite quarries, which called for granite top macadam, though local limestone at half the cost would wear nearly as well. Again, there are sections of the country where the high cost of cement, but the comparatively low cost of good stone or gravel, makes a concrete base for a pavement quite expensive, but a crushed stone or a natural gravel rolled base would cost less and be nearly, if not entirely, as durable. Of late years, too little attention has been paid to the macadam base as a foundation for a durable pavement. The public has been educated to think that concrete is the only possible foundation. The situation is similar to that of the septic tank in sewage disposal. Many public officials have heard and believe that such a tank turns sewage into drinking water.

In those parts of the country in which there has been much road work and improvement, it is much easier to get competent foremen and good labor than in sections where little, if any, of such work has been done. The result of good workmanship is very apparent. The local officials, also, are better educated in these matters and more inclined to give an engineer sway.

It should be remembered, also, that the character of the highways, whether city or country, must depend largely on the ability of the tax-payers to pay for them. A sparsely settled region may be very desirous of having good roads and be willing to be taxed for their construction, within reason, but Wisconsin, for instance, is far less able to pay $10 000 per mile for highways than is Massachusetts. The road mileage depends only to a limited degree on the population.

There can be little question of the benefit derived from the construction of good highways, but the degree of immediate benefit should have much to do with the selection of the material for such improvement. It might be, and frequently appears, that the benefits in the long run from a thoroughly high-class permanent construction will, theoretically, far exceed those derived from any cheaper construction. Because of the large immediate benefit received from the cheaper construction,

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however, and because the tax-payers are unable to pay for the more expensive construction, it may be much better finance to put in the cheaper highway, and thus not over-burden the community. Future development and growth of wealth may provide equitably for the construction of a better improvement. Is it not the same principle that governed the construction of our early railroads? It is a real recognition of this principle, not altogether possessed by some engineers, that has justified in many cases the voting of long-term bonds for short-lived improvements. Such a principle must be used with caution, however. It cannot be defended in New York or Illinois, but certainly can in Texas and Oklahoma.

In addition to the various points noted thus far, the esthetic features of the wearing surface of a highway should not be forgotten. In many cases, this feature should be made a very large factor in the selection of the proper material. Thus, a smooth, well-built brick pavement might be nearly as noiseless as one of bituminous concrete, and might have a much less maintenance charge and be more sanitary; but, along a scenic highway where the adjoining country is very highly developed esthetically, such as in the vicinity of a large city where there are many great estates, it is quite probable that the bituminous concrete pavement would be considered more suitable, pleasing, and satisfactory by those who pay for it. Such a road would be used largely by automobilists and comparatively little by truck gardeners. If this highway were lined by truck farms, however, could there be any doubt that the selection, if made between the two, should be brick, especially as it has been demonstrated that the bituminous concrete does not withstand iron-tired traffic? This selection, moreover, would be the correct one, even if the pleasure traffic was very large.

One of the points which it is believed has been neglected in the past is the relation of traffic to roadway width, especially where traffic is dense. It seems to the writer that it is possible to establish a logical, if empirical, formula which would be a considerable help at times in the solution of this vexatious problem. Some years ago, in preparing a report on a popular scenic highway near Chicago, a traffic census was taken at various points, and in addition statistics on other popular park highways in Chicago were obtained. These were plotted, together with the various existing roadway widths, and several curves tried on the same sheet. In this manner, it was finally decided that a parabolic curve having the equation:

$$Y = \sqrt{\frac{X}{6}} + 16,$$

would express the desired relation. In this equation, Y = the roadway width, in feet, and X = the number of vehicles.

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Thus, if it is determined from a traffic census that within 5 or 10 years a highway will have a probable traffic of 2 000 vehicles, the formula would give a roadway width of 34 ft., as shown on Fig. 1.

The diagram, or formula, is of course faulty, as it takes into account only two factors, but it has been a help in arriving at conclusions at times, and is offered for what it is worth.

Attention would be called to the fact that, because it is decided that the roadway is to be of a certain width, say 34 ft., it does not follow that the entire road should be paved with an expensive material. Cuyahoga County, Ohio (Cleveland), has hundreds of miles of roads improved on one side with brick and on the other with plain macadam

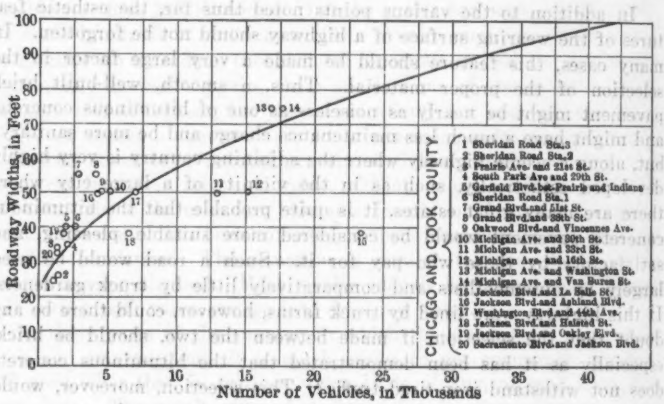

FIG. 1.

or gravel. The empty wagon and light horse traffic prefer the latter. Winona County, Minnesota, has paved the central 8 ft. of some of its highways with concrete, and provided wide macadam shoulders.

Mr. Thomes.

E. H. THOMES, M. AM. SOC. C. E.—An important factor in the selection of materials would be a standard specification for gravel, broken stone, etc., so that there would be no misunderstanding as to the sizes referred to; it would also be to the advantage of all buyers and sellers of broken stone. At present, this material is specified by minimum, maximum, and average lengths or diameters, by passing through a ring in any one or all directions, by the diameters of the screen perforations, by arbitrary designations, and in various other ways. Some designate No. 1 stone as the first course laid, which is usually the large stone, and others designate No. 1 stone as the small stone which first passes through the stone screens. Some refer to nut size, which may be any size between those of a hazel-nut and a

Mr. Thomes.

cocoanut. The speaker presented this matter to the Association for Standardizing Paving Specifications 3 years ago, and also in more detail to one of the road meetings of this Society 2 years ago, but very little has been accomplished since. His attention was called to this subject at the International Road Congress in London in the summer of 1913 by a report of the work done by the (British) Engineering Standards Committee on the standardization of road material. The report was published in the daily official journal of the Road Congress at London, but very little mention of that work has been made in the United States.

The Engineering Standards Committee is supported by the Institution of Civil Engineers, The Institution of Mechanical Engineers, The Institution of Naval Architects, The Institution of Electrical Engineers, and the Iron and Steel Institute, all of Great Britain. It has issued more than sixty reports and standard specifications, mostly along mechanical and industrial lines. In the summer of 1912 a Sectional Committee on road material was formed. It recognized the need and importance of the standardization of broken stone, and that was the first matter taken up. A sub-committee of the various interests was appointed, and, after considerable discussion and investigation, the British Standard Specifications for sizes of broken stone and chippings was adopted and issued by the Engineering Standards Committee in the summer of 1913.

Stones 1 in. in diameter and less are designated as chippings. There are four standard gauges of broken stone, $1\frac{1}{2}$, 2, $2\frac{1}{2}$, and 3-in. Broken stone specified as $1\frac{1}{2}$-in. gauge shall all pass through a $1\frac{1}{2}$-in. ring, and shall consist of the following percentages by weight: Not more than 15% passing through a 1-in. ring in every direction; not less than 65% more than 1 in., and not exceeding 2 in. in greatest length by measurement; not more than 20% greater than 2 in. in greatest length by measurement. The specifications for each $\frac{1}{2}$-in. larger gauge are similar to the foregoing, with $\frac{1}{2}$ in. added to each dimension stated, excepting the 3-in. gauge where 4 in. greatest length is used. Six standard gauges of chippings are recommended, namely: 1, $\frac{3}{4}$, $\frac{1}{2}$, $\frac{3}{8}$, $\frac{1}{4}$, and $\frac{1}{8}$-in. All 1-in. chippings must be capable of passing through a hole 1 in. square; and at least 70% by weight must be retained by a sieve having holes $\frac{3}{4}$ in. square. The specifications for the other sizes are similar, with the substitution of the sizes of sieves. With $\frac{1}{8}$-in. chippings, the sizes of square holes on which the material must be retained is $\frac{1}{16}$ in.; everything passing that sieve is to be called sand. They also specify gauge plates, sieves, and methods for testing fair samples of at least 100 lb. of the material. It will be seen that in Great Britain the broken stone is divided into more sizes than is customary or advisable in the United States. Some of the quarries there have been developed much longer; they are down

Mr. Thomes. deeper, and in sounder and more uniform rock. Some of them also furnish washed chippings, which are advantageous for bituminous work.

The materials and quarry practices in the United States are similar to those of Great Britain, and, with the work already done by the Engineering Standards Committee, it will be an easy matter to obtain a standard specification for sizes of broken stone. The question must be considered from the standpoint of both sellers and users, for all purposes. Conditions here seem to warrant that this subject be taken up by this Society, and in order to secure some definite action on this matter, the speaker will move that the Board of Direction be requested to appoint a Special Committee on the Standardization of Broken Stone, to report to the Society as soon as practicable.*

Mr. Whinery. S. Whinery, M. Am. Soc. C. E.—The diagrammatic outline of factors in road work submitted by Mr. Green is generally a very satisfactory classification of the elements entering into the design of highways. The speaker, however, would remove the element of esthetic considerations from the subdivision under "Character of Supporting Country" and give it a separate sub-head at the end of the second column. It may be questionable, however, whether this item should have any place in such a diagram, devoted as it is to what may be called the economics of highways. Esthetic considerations, of course, cannot be ignored finally, but, from the engineering point of view, they are foreign to the main problem, which, in highway design, as in other engineering structures, may be reduced to the question: What design will best serve the purpose intended, with due regard to the highest yield in returns on the money invested? Necessity, of course, may force us to substitute the question: How may the highest economical results be obtained for the money available? Esthetics is not a factor in solving either problem.

Nevertheless, it must be finally considered, and the answers to the foregoing questions may have to be modified accordingly. The highway should be designed along the same lines as any other industrial structure, as, for instance, a building for manufacturing purposes. Here the efforts of owner and architect will be first directed wholly to questions of utility and economy. These settled, the question of the adaptation of the design to artistic ideas or standards will be considered, and such modifications or additions as may please the designer, or as the owner may feel willing or able to afford, will be incorporated.

In any event, the question of esthetics, however important a factor it may be considered in designing city streets, boulevards, and park-

* This matter is referred to the Special Committee on Bituminous Materials for Road Construction, with the request that some action be taken to secure the standardization of broken stone in the United States.

ways, is usually so unimportant in as far as it relates to the roadway itself, in the case of country highways, as to be almost negligible. Mr. Whinery.

The speaker cannot agree with Mr. Green that the building of temporary, inadequate highways is justifiable, except in rare instances, especially where long-period bonds are issued to pay for them. In the present well-developed status of scientific highway construction and economics, and the possibility of securing capital for enterprises of real merit, the spending of the public money for highways of temporary life and usefulness is generally inexcusable. Mr. Green's citation of early railroad history does not present a parallel case. The temporary and inadequate railroads built in the early days resulted from the undeveloped conditions of the principles and requirements involved, rather than from financial or economic considerations. Intelligent and reasonable people do not pattern after them now, to the extent of adopting an ephemeral and clearly inadequate construction, which must obviously be abandoned and reconstructed in a few years. It would be more difficult to finance such an enterprise than one properly designed to serve adequately the prospective requirements of the reasonably near future.

As a business proposition, the speaker does not believe that it is wise or justifiable to issue long-period bonds to pay for a structure the useful life of which will be but a small fraction of the period the bonds are to run. In other words, we have no ethical right to saddle upon posterity a burden from which posterity will not be benefited. From the sound business standpoint, a highway improvement the benefits from which will not redeem its cost during its life, is not a justifiable investment for a community or a State.

This, however, is not an argument in favor of unnecessary or reckless expenditure in the building of a highway. No more money should be expended on it than the conditions or needs of a community, or the general public, now economically justifies or is likely to justify during the life of the improvement. The strict application of this criterion would rule out the costly improvement of many ordinary highways where a properly constructed dirt road is all that the conditions warrant.

The considerations relating to selecting the type of roadway and the materials to be used in its construction, submitted by Mr. Green, are in the main apropos and sound. It would have been appropriate for him to add a warning against the use of materials, merely because they are of local occurrence, or because of their cheapness, and a strong protest against the use of materials or methods the utility of which has not been sufficiently established by experience. The use of experimental materials and methods, except on a very small scale for trial, ought to be discouraged by the prudent engineer. Progress demands, of course, that promising innovations from established practice shall

Mr. Whinery.

be tried out, but the conservative engineer will prefer to let other people do the experimentation, except on a scale which will not result in any considerable loss to his clients, in case of failure.

Personally, the speaker feels that, in road and pavement work, we ought to adhere pretty close to materials and methods the value of which has been clearly established.

The more important country highways are now subjected to very much the same conditions of use as the average city street. Long experience, both in America and abroad, has, after thorough trial and much foolish and often disastrous experimentation, determined that certain types of pavement and certain materials, can alone be depended on to produce satisfactory results, and their use on city streets has become standard. Among these the choice, of course, will be influenced by local conditions. Prudence and good engineering would seem to dictate that we should apply this experience and established practice to the construction of modern roads. The common argument that the first cost of a standard roadway pavement is too great to warrant its use is in most cases fallacious or untenable. An established highway is, nearly always, a permanent structure, and should be designed with reference to its true economy, that is, its value as an investment. If, for instance, a standard brick-paved roadway will last 20 years where a water-bound macadam surface will last but 5, and the former can be maintained at a less annual cost, it will be, at ordinary prices, much the more economical in the end, and the engineer should make this plain to his clients. He should vigorously discourage the common disposition of the public to waste its money on temporary highway improvements because of their low first cost, and should encourage the building of roads in such a manner as will prove most truly economical. These are such trite arguments that they seem to need to be repeated until their truth is accepted and more generally acted on.

The speaker agrees with Mr. Green that the width of the paved roadway should be determined with reference to the density of travel to which it is likely to be subjected, but thinks that the tentative formula submitted for determining the proper width, when applied to country roads, should be used with even more caution than the word tentative implies. The data on which it is based seem to have been derived from conditions on very heavy traveled streets, boulevards, and suburban roads, where the conditions are very different from those on most country highways; but, even in the examples used, there is such a wide variation in the figures as to suggest their uselessness as a basis for determining a standard of width of roadway. Thus, as nearly as can be determined from the diagram (Fig. 1), the number of vehicles per foot of width of roadway varies from about 35 to more than 600.

It is doubtless true that the higher figure indicates dangerous crowding, but, even so, a roadway having only 6% as many vehicles per foot of width cannot be regarded as an appropriate factor in determining a proper and safe standard for allowable density of travel. Mr. Whinery.

If the proper width is to be determined by this method, only observed data, from such highways as seem to have just about the quantity of travel they can safely and conveniently accommodate, should be used. Deductions or formulas from such observations would be far more useful and trustworthy than the theoretical considerations and conclusions that some have tried to work out and apply. It is very desirable that data from highways of the character mentioned should be collected and collated.

W. W. Crosby, M. Am. Soc. C. E.—Mr. Green is to be complimented on his interesting presentation of this subject and on his exposition of the factors involved in the selection of the materials or the methods of construction. His diagrammatic outline of factors in road work is most interesting, and if one will take the trouble to compare this diagram with that submitted by the speaker 3 years ago,* an interesting illustration will be afforded of the ideas in common as well as those of difference which may be held by two individuals. Mr. Crosby.

The speaker has been particularly interested in the scientific solution, proposed by Mr. Green, of that troublesome problem, the determination of the proper width of a roadway, and desires to express his belief that it is this sort of treatment of many of the present questions in highway engineering that will do most to advance the status of this particular branch of the profession and to erect for its future growth a substantial foundation.

In reference to this matter, and with the hope of furnishing data for the basis of such treatment of some of the problems connected with the selection of materials and methods in highway work where the traffic factor seems preponderating, the speaker offers the following concerning traffic records in general and in a particular instance coming under his observation:

In 1903 he urged highway engineers to take traffic censuses and study the relations between traffic conditions and the life, or expense for maintenance, of road crusts. He also suggested that from the study of such relations much might be learned that could be used advantageously in making selections for construction under known or determinable traffic and other conditions.

The Illinois Highway Commission began this traffic census in 1906. In 1909, the Massachusetts Highway Commission took a census of traffic on Massachusetts roads.† Other traffic censuses have been taken

* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 6.

† Report, Massachusetts Highway Commission, 1909, p. 128, etc.

Mr. Crosby. and reported by officials and individuals from time to time since these dates, and though at first there was some skepticism as to the value of such investigations, the speaker believes that there is now a fair amount of general agreement that, with proper deductions made therefrom, they are of great value. He also believes that there is general agreement now that a road crust satisfactory and economical for sustaining rubber-tired motor traffic may not be so, either for horse-drawn or for hard-tired traffic, and *vice versa*, as he has stated previously.

During the summer of 1913, in order to permit the improvement of a highway by the State Roads Commission of Maryland, the Board of Park Commissioners of Baltimore opened to all kinds of traffic a park road paralleling the Reisterstown Turnpike for a distance of 2 500 ft. Previously, this park road had been kept in excellent condition by the maintenance, on the quartz-schist macadam, of a bituminous (or "pitch") surface, at an annual cost of about 9 cents per sq. yd. Within 90 days after the opening of the road to all kinds of traffic, this surface or "carpet" became in dry weather a dusty mass of loose material and in wet weather a mess of slimy black mud an inch or more thick. Depressions in the macadam beneath also began to show soon afterward, and by December, 1913, the road crust appeared to be generally going to pieces. The quartz-schist had shown a rattler test of about 4, though its cementation power, in use, had proved to be high—probably due to its iron content.

Southward from this 2 500-ft. stretch, the pleasure traffic continued along the main road through the park, the commercial traffic, which consisted mainly of horse-drawn, hard-tired vehicles and motor trucks, being turned out of the park at this point. The traffic on the 2 500-ft. stretch before its opening to commercial vehicles was therefore equal to that now using the road southward from this stretch, as shown by the census under "B" in Table 1; the combined traffic on the 2 500-ft. stretch is that shown under "A".

The carpet referred to in this case was made up by successive applications, in 1911, 1912, and 1913, of "75% Asphalt Oil", the layer of each being covered with pea gravel or stone chips, and at the time of its first subjection to the new traffic it was about ¼ in. thick on top of the macadam. The last application of bituminous material (or "pitch compound") and stone chips was made the day before opening the road to mixed traffic. The width of the carriageway, exclusive of gutters, is 22 ft.

The figures of the traffic census taken in 1904 are inserted in Table 1 in order to give an idea of the increase or change in character of the traffic between that time and the present. They also give some information as to the amount of traffic under which a water-bound macadam of comparatively soft broken stone may be maintained satisfactorily at a not excessive cost. Such a road crust in this instance

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required treatment with a pitch compound before the traffic reached the figure in "A", in order to permit its maintenance with satisfaction at any cost.

TABLE 1.—TRAFFIC CENSUS ON MAIN ROAD IN DRUID HILL PARK, BALTIMORE.

Classification.	Maryland factor.	November 24th to 30th, 1904.		November 20th, 1913.			
				"A"		"B"	
		Average per hour.	Units.	Average per hour.	Units.	Average per hour.	Units.
Saddled horses	1	3	3	10	10	0	0
One-horse vehicles	2	28	56	25	50	7	14
Two " "	4			35	140	0	0
Three " "	6	10	150	1	6	0	0
Four " "	8			0	0	0	0
Six " "	12			1	12	0	0
Total horse traffic		41	209	72	218	7	14
Bicycles	2	2	4	8	16	6	12
Motor runabouts	10	0	0	9	90	7	70
4-seat motor cars	20	1	20	43	860	39	780
6 " " "	40	0	0	37	1 480	35	1 400
Motor trucks	20	0	0	9	180	0	0
Totals		44	233	178	2 844	94	2 276

In making deductions from the data in Table 1, it may be of interest and value to refer to the similar data and the conclusions thereon,* furnished by the speaker a year ago concerning the Park Heights Avenue experimental work.

One further point may be indicated. It will be noted from the traffic tables that the increase in traffic on this section, due to the admission of commercial vehicles, was 204 units of horse-drawn and 364 units of motor traffic, the latter being composed mainly of motor trucks. Both these types have a high abrasive or crumbling effect. Evidently, the thin carpet over the soft macadam stones was unequal to the task of maintaining its integrity under this traffic, or of absorbing the shocks coming on it sufficiently to prevent the disintegration of the stones under it. Hence, under this traffic, the stones were abraded or broken to pieces, the bond of the carpet to the stable foundation was thus destroyed, and the carpet itself aided in going to pieces. The actual destruction proceeded along these lines, the carpet first scaling off in spots and revealing in these places a ground-up condition of the macadam surface beneath. Then the scales of the

* *Proceedings*, Am. Soc. C. E., for September, 1913, p. 1702 *et seq.*; or *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 188 *et seq.*

Mr. Crosby. carpet, thus loosened, disintegrated, and finally the whole carpet went into dust or mud.

The experience here, as well as in some other cases with similar conditions coming under the speaker's observation, is quite similar to many instances of the failure of thin carpets on concrete roadways, and supports the theory that, with friable bases, the carpet must have such thickness and character as will prevent the disintegration, under the shocks of traffic, of the surface of the base itself, if successful maintenance is to be had.

Many have probably observed the development in bituminous pavements, under traffic amounting to more than a certain minimum, of waves or corrugations approximately at right angles to the line of traffic, and that these waves, where they occur, have an appreciable regularity. This regularity has probably much to do with the suggestion which has been made that such waves be referred to as "harmonic waves" to distinguish them from the more isolated humps and hollows which result from an excess of bitumen in the pavement at one side, or from a settlement of the pavement within a small area, or from something of that nature. Indeed, the theory has been advanced that the harmonic waves referred to may be caused in many cases by "harmonic percussion", such as may be set up when a moving heavily-loaded vehicle supported on flexible springs strikes an obstacle in its path and attains in its mass a certain rhythmic vibration. This vibration, transmitted through the wheels to the road surfaces, probably produces successive light and heavy stresses in such surfaces, especially when the vehicle is self-propelled. The speaker does not feel that the theory of harmonic percussion is a solution for all harmonic waves, but merely that it is one possible solution of them under certain conditions. He has noticed the appearance of such waves adjacent to an obstacle in the path of traffic when they were not visible away from the obstacle, although the road surface and the road traffic were everywhere, within a reasonable radius, identical. On the other hand, they appear to occur at times when no obstacle such as would set up harmonic percussion is apparent, and in these cases it is quite probable that they are due to excess of bitumen in the road surface, or to improper grading of mineral aggregate for the traffic conditions to be met, or to an improper character of the bitumen, or possibly to some other causes. The speaker thinks it would be of considerable interest, if there could be some discussion by engineers concerning the reasons for the formation of the harmonic waves referred to, with such observations and relation of facts connected therewith as their experiences will permit them to make.

Mr. Brooke. MARK BROOKE, ESQ.* (by letter).—There is no one best paving material for all conditions, and the necessity of carefully considering

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in each case the many factors shown with such definiteness and detail in Mr. Green's diagram cannot be emphasized too much. As Mr. Green has stated, the use of that diagram in practice is comparatively simple, for it will be found that the choice in many cases will be restricted either by limitations of first cost or by local or general conditions which automatically cut out certain materials from consideration. Mr. Brooke.

In contradistinction to the bridge engineer, for example, the highway engineer often finds that the sum of money placed at his disposal is not based on his own estimate of a definite project, but that he is allotted, in a general distribution of funds based on anything except a careful estimate or budget, a lump sum of money which he must spread over a certain section to the best advantage of the entire area. The question is then, not what is the best material, but how much he can afford to spend on a given street or road, so that the problem is apt to resolve itself into a selection of materials for the most suitable pavement which can be constructed for a definite limited sum, say, for $1.25 per sq. yd. Even in this narrow field of choice careful consideration should be given to all the items in the diagram, as far as the limitation of cost will permit.

Again, it will be found possible to simplify the selection of materials by applying once for all such of the factors in the diagram as have a general and permanent application in any given locality, thus eliminating a large class of materials, and restricting the choice to a few types.

In Washington, for example, as a result of experience and of consideration of the various factors such as first cost, character and volume of traffic, suitability of local material, esthetic considerations, etc., the problem has been reduced to a selection of one of three types of asphalt pavement for city streets, and bituminous concrete, cement concrete, or water-bound macadam with a surface treatment of light tar or oil, for suburban streets and county roads.

In the city practically nothing is used but a standard sheet-asphalt, a very limited quantity of 2-in. asphalt block, and an increasing quantity of bituminous concrete in accordance with the specification of the department.

This bituminous concrete, which is very similar to the various pavements of that type laid between 1870 and 1880, appears to have all the advantages and but few of the defects of sheet-asphalt, and is cheaper.

A gravel cement concrete base is now being used under all pavements, except in a few instances where old macadam roads have been surfaced with bituminous concrete.

Macadam, especially an old macadam road, will make an excellent base which will outlast any bituminous wearing surface yet devised,

Mr. Brooke. but it cannot be kept free from local settlements, nor can it be easily or economically resurfaced. The man who lays bituminous concrete on a macadam base should realize that he is storing up trouble for himself or his successor 15 years hence.

Whether or not it is advisable to invest in the more expensive base to reduce the cost of future resurfacing is, of course, a question every one must decide for himself.

The engineers in Washington for some time past have been of the opinion that it is better, as a general practice, to lay a base which will not have to be destroyed every time a new surface is put on it, and hope to relieve their successors of the extra expense and difficulties encountered during the last 10 years in resurfacing pavements laid on macadam and bituminous macadam bases—types of construction which conformed to the best engineering practice of the time, when the use of Portland cement was still in its infancy.

For 30 years a concrete base has been laid without either longitudinal or transverse joints, with no significant bad results, and the writer doubts very much the necessity of the elaborate and costly provisions for such construction which are so often seen in cement road work. On some of the cement pavements in Washington, no joints whatever have been provided, on others there is a thin transverse joint every 50 ft., made by bringing the concrete up to a vertical face, laying a strip of folded tar-paper against this face, and then depositing the next batches against the paper. Thus far, no difference in the action of the two classes of construction has been observed. If any joint is used it should be as thin as possible. A wide joint is a weak joint, and its partial contraction will invariably force the filler into a pronounced ridge, as objectionable to motor traffic as any thank-you-ma'am.

When this pavement is laid with a curb it is kept about 1 in. below grade, so that, should the thin bituminous carpet with which it is covered prove an uneconomical form of wearing surface, the pavement can be used as the base for a thicker surface of bituminous concrete and still have sufficient face of curb showing.

For many years trap rock has been the standard material for water-bound macadam. Recent experiences in the oil and tar surface treatment of trap and limestone roads indicate that, for moderate traffic at least, the surface treatment does as well with the limestone as with the trap, and the saving effected by using limestone about offsets the first cost of the surface treatment.

Referring to the practice mentioned by Mr. Green, of paving the center 8 ft. of certain roads with concrete and providing wide macadam shoulders for the light traffic, the writer believes it is a mistake not to make the paved strip wide enough to carry two lines of traffic. If this is done, it is unnecessary for passing vehicles to cross the joint between the concrete and the shoulders—the weakest part of the road—

a condition which would be particularly objectionable at seasons when the earth shoulders were unsuitable for traffic, and all vehicles would seek the paved portion of the road. The engineers in Washington have had this experience in the case of a trunk-line road paved with granite block, the center 8 ft. of which was laid with asphalt for the benefit of light traffic. As a matter of fact, all traffic, both slow and fast, tries to get on and stay on this 8-ft. strip, thus increasing both the congestion of traffic and the wear on the asphalt. Mr. Brooke.

The absence of any numerical weights assigned to the factors in Mr. Green's diagram appeals to the writer as being most sensible. The relative values to be given these factors in selecting a material must be different for each case, and a table which attempts to fix values for general application is apt to be misleading.

Materials and methods successful in one case are failures in another, and the lack of knowledge or realization of the difference in conditions in the two cases often leads one to draw general conclusions which are not warranted.

To attempt, with present knowledge, to solve road problems by the use of general formulas or tables is, as a well-known highway engineer has described it, "simply holding at arm's length work which should be brought close to us".

By careful study and accurate records of failures and successes, by standardization of specifications and tests, and by uniformity in methods of taking traffic censuses, one can largely eliminate the elements of personal taste and judgment in the consideration of the various factors under discussion, and still further simplify and standardize the selection of materials.

George W. Tillson, M. Am. Soc. C. E.—This subject is of great importance; it has been discussed for a great many years, and will be discussed for many years to come. The speaker has made the statement several times that, in the preparation of plans for road and street work, engineers were more behind than almost any class of constructive workers. That statement will have to be modified from time to time, as the subject is receiving so much discussion from year to year and from day to day. Mr. Tillson.

It will be remembered that Mr. Green started out with a proposition that what was said in the discussion must be new. That is a very difficult requirement, and, under it, the speaker feels that he cannot qualify, especially after listening to Mr. Green's discussion; he thinks that a man who could talk for ten minutes and say nothing but what was new to every member would be a wonder. If the speaker can suggest one idea that is new, he will be satisfied. The point he wants to make is that the ideas which govern roadway and street construction and the design of both must not be said once and then laid aside. In order to make inspectors and road supervisors entirely famil-

Mr. Tillson. iar with them, they must be reiterated, told over and over again, said in a new way, or said in the same way, until they know them as thoroughly as they know the alphabet.

In the design of work of this kind, it seems that the first thing an engineer must do—and the whole situation will be discussed in as general a way as possible—is to make himself thoroughly familiar with the work that is before him. If an engineer is to design a bridge, if an architect is to design a building, the first thing to do is to find out what will be required of this bridge or this building. The first thing the engineer must do is to study the requirements of the road he is to build or the pavement he is to construct.

In doing that, he has a great many things to consider. That is brought out to a great extent by Mr. Green. The character of the traffic: Until the last few years no one has considered it necessary, or, if necessary, that it was worth while, to have a traffic census, either in city streets or country roads. No longer ago than December, 1912, at the Road Convention in Cincinnati, after an elaborate paper on "Traffic Census," and what was to be gained by it, a State engineer said, "Well, after all this traffic census, you have got to use your common sense." That is true. That is the very object of having the traffic census, so that common sense may be used intelligently.

Now, the traffic is not the only thing that must be studied. One must study where the road is to be laid, what its objects are, the character of the different traffics, and, another thing, not only the entire traffic, but how that traffic is to be applied.

The engineer of the Borough of Fulham, London, has calculated that, under a certain traffic which has been observed, a wood pavement will last for a certain term of years; and that, when a wood pavement is laid on any street, he can calculate its life in accordance with the traffic which is to be applied. That must be considered to a great extent in a way which the speaker will mention later.

Now, after one understands fully the requirements of the road, the next thing is to find out the character of the materials available for use. This part of the question was not nearly as important 30 or 40 years ago as it is to-day. At that time, stone in its various forms was practically the only material used in road or street construction. Now, there is not only stone, but brick, wood, and the bituminous materials, the latter laid in all their various forms and composed of the different kinds of bitumens.

The engineer, in studying this phase, must consider first the character of the stone. He must know what the different kinds of stone will stand under the different kinds of traffic. The speaker was severely criticized once in college when he said—in considering the abstract question of foundations—that stone made the best foundations. His professor contended that the statement was absolutely wrong, because,

under certain conditions, sand is just as good a foundation as stone. Now, under some conditions of traffic, a cheaper stone is just as good as a dearer, so that, though in a certain way one stone would be better abstractly, the other stone might be just as good concretely. Mr. Tillson.

It is the same way with other materials. The engineer must understand brick and the different grades of brick; he must understand wood, what the different varieties of wood will do, what kind of treatment the wood requires; and then, when it comes to the bituminous pavements, he is uncertain at the present time. It does not make any difference where he goes, he will get different advice, different information. The information may be true, for the reason that different people are studying different kinds of bitumens, and they know more about the bitumens that they are studying than about any others, and while they are giving perfectly good and true information about their own, it seems to be contradictory to what was received before, because they did not understand the values of these other bitumens.

Now, let it be assumed that the engineer understands thoroughly the condition of his road, and all the different qualities of the materials, both in their crude state, and also how they can be mixed in order to produce satisfactory results.

The next thing is to make the application, and here there should be elaborated what was said about the traffic and the wearing out of the pavement in a certain length of time. Take, for instance, the assumption that a traffic of 60 000 000 tons will wear out a pavement, and the road will receive 2 000 000 tons of traffic per year. The logical deduction would be, then, that the pavement would last 30 years. Not necessarily; because there are certain materials in use which are affected chemically by climate and by the natural action of the atmosphere.

Now, as was stated by Mr. Green, wood and the bituminous pavements do require a certain amount of traffic in order to give the best results. So that, even if there exists a traffic which, theoretically and mathematically, would wear out the pavement in 30 years, this traffic may be so scattered that it would not be such as would be most economical and best for the pavement; and, on account of climatic conditions, it might be destroyed by disintegration and decay before it was worn out. Brick and stone are probably the only paving materials now in use the life of which can be determined according to the foregoing principles, unless the traffic is so heavy that it will wear out the pavement before the materials decay or become disintegrated. The best illustration of this is in the use of wood. If these 60 000 000 tons of traffic will be applied to a pavement before the wood would naturally decay, there is no necessity for treating the wood with any material that will prevent it from decaying. That principle is recognized in Europe, where, in the great majority of cases, the wood will wear

Mr. Tillson.

out before it will rot out, and it is only superficially treated. In Paris, where they use soft wood, and where the wear of the traffic in certain streets is such that the pavement will last only 7 or 8 years, they use what might be called no treatment; that is, the blocks are immersed for perhaps ½ hour in creosote oil and then laid in the streets.

Paris, last summer, was erecting a plant in which the blocks will receive a treatment of 10 or 12 lb. per cu. ft. That is not necessarily to keep the blocks from decaying, but to prevent them from swelling and causing the pavement to bulge. That is the main thing that they hope to gain by increasing the quantity of preservative to 10 or 12 lb.

It can readily be seen that it would be wrong to use an elaborate form of treatment to prevent decay when the block itself will wear out before it would rot out.

In America, where there is not this intensive traffic, and where a pavement is expected to last longer, it is an entirely different proposition. Take a certain street, for instance, in Brooklyn, where wood block has been laid about 10 years. Practically no repairs have been made, and if the traffic continues as it has in the past 10 years—and being a residential street, it probably will—that pavement will not be worn out in 50 years from the time it was laid. So it will be seen how necessary it is to lay treated blocks under such traffic, so that they will last as long as they can, as far as decay is concerned; and for that reason it makes a difference whether the amount of traffic that will wear out a pavement is scattered over 4 or 5 years or 25 or 30 years.

There is just one little point more, which was referred to by Mr. Green and also to a certain extent, by Mr. Whinery. All these different principles that have been spoken of can be settled in the laboratory; but, if an engineer determines on granite, for instance, for a certain street, and writes to his client, who may be 400 or 500 miles away, and tells him of his decision, the client might write back and say there is not a granite quarry within 1 000 miles. The point to be recognized is the question of availability. That is of the utmost importance, and the speaker has often thought that, in the possibility, or rather the power, of Nature to furnish everything necessary for the development of the world, the development of civilization has been shown in the pavement line as much as in any other way.

In the Central West, where there is no rock suitable for pavement, where all outside materials can only be brought in at great expense, Nature has furnished a material—clay—from which can be, and is, manufactured a brick that gives satisfaction and is almost as durable as the best of stone itself.

In summation, let it be said that if an engineer, who has roads or street pavements to plan, has first a knowledge of the conditions which his pavements must meet, if he understands thoroughly the qualities of

Mr. Tillson.

the available materials, and if with that knowledge he makes his application in accordance with sound common sense, taking into consideration the atmospheric conditions and the availability of materials, he will undoubtedly reach a conclusion that will give satisfactory results.

The following question has been asked: "What conditions would be considered, and would govern, in making a selection of asphalt blocks, brick, or bituminous concrete for a street, say, in Brooklyn?" This is easily answered, because, as a matter of fact, Brooklyn uses only one of the materials mentioned, *viz.*, asphalt blocks. Brick has not been used because it is expensive and makes a more noisy pavement than asphalt, so that the latter is more desired by the property owners. Asphalt blocks are generally selected in preference to asphalt where the grades are comparatively steep. The blocks are less slippery than sheet-asphalt, as they are made of coarser materials, and on account of the joints between the blocks. Brick would be selected for a street on which the traffic would be too heavy for asphalt block. The speaker has never used asphaltic concrete, and cannot give an opinion as to whether this material would give a better wearing surface than asphalt blocks.

Mr. Meeker.

ROBERT A. MEEKER, M. AM. SOC. C. E.—The speaker cannot accept the theory of harmonic waves, said to be due to traffic, nor the beautiful system of harmony of the spheres, for, from practical experience, it has been learned in his Department that it is very necessary indeed to watch the output of the mixer carefully. The proportions of stone, of sand, and of asphalt may be all right, but the first run of the mixer will be too rich in asphalt, the middle of the run will be just right, and the latter part of the run will be too lean; consequently, one would have a pavement composed of, for example, 12%, 8%, and 4% of asphalt, all from the same mixer, notwithstanding that the stone was weighed and measured carefully, as was the asphalt, before either was put in the mixer.

This mechanical failure is often overlooked. It caused so much trouble that the speaker was led to make a careful investigation of the run of the mixer, and it was found that, in many cases, the asphalt concrete which had been condemned as not being properly mixed, owed its lack of uniformity, not to the workmen who were in charge (they had done their duty faithfully and carefully), but to the machine.

In connection with this theory of harmonic waves, there was one very important point made by Mr. Washington which is worthy of more than passing notice, namely, the irregularities caused by improper rolling. The speaker has had considerable trouble in the past, even with water-bound macadam roads. When the causes of the so-called waves are carefully analyzed, it will be found that the trouble lies,

Mr. Meeker. not in the traffic, but in faulty construction. The first macadam roads built in the United States were rolled with heavy three-wheeled rollers. There never were any waves in those roads. Later, an endeavor was made to reduce the cost of construction, and, to that end, horse rollers were substituted for the heavy English steam rollers; still later, different types of steam rollers were used, and, last of all, the tandem roller was brought forward as the best. The speaker tried them, and had macadam roads with harmonic waves long before automobiles were used. The real reason for these waves, or hills and hollows, in the road, may be found in the method of construction. If the base is not thoroughly compacted, not thoroughly rolled, and not brought to an even and uniform surface, then, if those waves do not appear at once, they will develop eventually. When to this wavy base is applied bituminous concrete which is not uniform in composition, two sets of waves—two sets of hills and hollows—will appear, not due to traffic, but to careless construction.

Highway engineers should not try to dodge their responsibility for poor workmanship by charging it to motor vehicles or to the rhythm with which they travel over the highways, when the fault lies in the manner of construction. This must be carefully watched. Careless construction was one of the reasons for the failure of so many penetration roads. The speaker has had experience with penetration roads in which he was told that the distributing wagon had stood for a time at a certain point and that the hump of bitumen, seen on top of the road, was due to the fact that the controlling valves were leaking.

In one case, coming under the speaker's immediate observation, he knew that the wagon had not stopped within 100 ft. of that particular point; therefore, this explanation was evidently wrong; hence, a series of investigations was made, whereupon it was found that the percentage of voids in the road varied greatly. Where the road was comparatively dense, the bitumen appeared in humps on top of it and disappeared entirely at a depth of 2 in., and where hollows appeared it was found that the bitumen had penetrated in many cases to the depth of 7 in. The same quantity of bitumen per square yard of surface having been applied, there was consequently a recurrence of the fat and lean bituminous mixture encountered in the output of the mechanical mixer. A careful analysis of numerous faulty spots, in both broken-stone and bituminous pavements, has convinced the speaker that the varying percentage of voids is a fruitful cause of many so-called failures; in other words, a lack of homogeneity in the mass is the real cause of the wavy surfaces so often seen.

Mr. Beaty. R. E. Beaty, Assoc. M. Am. Soc. C. E.—These remarks may not have a literal application to the discussion of "Factors limiting the selection of materials and of methods in highway construction", but,

with regard to the use of wood paving blocks, the speaker is of the opinion that some mistaken ideas are abroad which should be corrected. Mr. Beaty.

In European practice it has been customary to use a very light treatment in preparing blocks, that is, to treat them with 10 lb. of oil per cu. ft., or in some instances merely to dip them into the preservative.

The wood principally used in France is known as "Landes" pine; in England "Baltic deal" is used. An examination of some of these blocks shows very uniform wear, and that the wood very much resembles southern short-leaf, yellow pine, with the difference that the wood is much more nearly uniform in density and strength of fiber.

Tests made by the Borough of Manhattan, and by the Department of Forestry, of the United States Department of Agriculture, show very clearly that although some American pines very much resemble the timber used in France and England, they are of much less uniform quality. The characteristics of southern pines will be considered in more detail later.

In the speaker's opinion, it will be taking a grave risk to try to follow the European practice of using a light treatment of oil. This method will have the effect of leaving some of the blocks in the pavement untreated. This condition is indicated to some extent in foreign countries, where numbers of blocks have to be removed on account of rotting.

All the speaker's observations of experiments and—what is of more value—of the actual condition of 250 000 sq. yd. of blocks after treatment, in charges of from 400 to 500 sq. yd., has convinced him that, as the quantity of oil is decreased below 20 lb. per cu. ft., it becomes increasingly difficult to get either a uniform distribution of oil between the individual blocks making up the charge, or a satisfactory penetration into all the blocks. This difficulty can be overcome to some slight extent by the slow application of the oil, but this is not commercially desirable, for the reason that lengthening the time of treatment would often curtail the output of plants fully 33 per cent. Attempts at light treatment have been observed many times. In one case, 16 000 sq. yd. were being treated with 16 lb. per cu. ft. The specific gravity of the oil used was 1.06 at 38° cent. At first a 16-lb. treatment was tried, but it was found necessary to increase this to 18 lb., because many of the blocks, when the lighter treatment was applied, showed a penetration of only ¼ in. from the surface. The inference is, that in injecting 16 lb. or less into the blocks, the duration of time the pressure is applied to the treatment cylinder is too short to allow the pressure to be raised to a height sufficient to cause oil to penetrate the denser and harder blocks before all the oil is used up. In experiments reported in December, 1912, on various samples

Mr. Beaty. tested for the Department of Public Works, Manhattan, it was shown that the quantity of oil taken up per cubic foot by the blocks, under a pressure of 150 lb. per sq. in., which pressure was necessary to inject an average of 20 lb., varied widely among the different species of southern yellow pine blocks making up the samples. Some blocks received the equivalent of 28 lb. per cu. ft., and others received only 12 lb. At less than 50 lb. cylinder pressure, 10 lb. can be injected. If thus treated, many blocks in a charge would receive practically no oil. This lack of uniformity in material, though not so marked in the "Landes" pine and "Baltic fir", indicates a cause for decay in individual blocks.

There is in very general use, by some wood block manufacturers, the term "Southern Yellow Pine" to describe the timber used in manufacturing their product. This nomenclature, as used in the 1913 *Proceedings* of the Association for Standardizing Paving Specifications, is intended to cover all the five or more varieties of pine found in the South, and it was so stated in the "discussion" at the time of their adoption. That the term is misleading, indefinite, and inaccurate is readily seen by a reference to Bulletin No. 13 of the Division of Forestry, of the United States Department of Agriculture, in which five species of pine, several of them occupying large and definite areas of the Southern States, are given, as follows:

"Long-leaf" pine (*"Pinus palustris"*, Miller);
"Cuban" pine (*"Pinus heterophylla"*);
"Short-leaf" pine (*"Pinus Echinata"*, Miller);
"Loblolly" pine (*"Pinus taeda"*, Linn); and
"Spruce" pine (*"Pinus glabia"*, Walt).

These species vary widely in physical characteristics, such as average percentage of resin, number of annual rings, weight per cubic foot, average percentage of heart wood, strength in compression of fiber parallel to the grain, etc. Two of these definite groups will be compared: the "Long-leaf" and the "Short-leaf" yellow pine. Tests by the U. S. Department of Agriculture showed the following relative values:

	Long-leaf.	Short-leaf.
Strength in cross-breaking......	10 900 lb.	9 230 lb.
Strength in compression (per square inch):		
Average lowest...........	5 650 "	4 800 "
Average highest...........	6 850 "	5 900 "

In summarizing the results of tests on various pines the Department expert states: "From these results, though slightly at variance,

we are justified in concluding that Cuban and Long-leaf are nearly alike in strength and weight, and excel Loblolly and Short-leaf by about 20 per cent." Mr. Beaty.

These values were checked by experimental tests, at Columbia University, on samples secured by the speaker for the Borough of Manhattan, to which reference has previously been made. The results of these tests are not given here, as the tests were made to demonstrate the value of a requirement for a minimum number of annual rings in timber used in the manufacture of blocks. It is interesting to note, however, that the average strength of three samples of long-leaf was more than 10 000 lb. per sq. in. in compression, parallel to the fiber, and some samples of short-leaf were as low as 5 600 lb. per sq. in.

Other tests, made at this time by the Highway Department, indicated clearly that "short-leaf" pine, as compared with "long-leaf", has a greater range of expansion and contraction, and is more difficult to impregnate uniformly with preservative. Each of the species considered has its uses, but, as they often vary widely in price, as well as in physical characteristics (as has been shown), then, for the mutual protection of the bidder and the ultimate user, engineers should specify clearly the kind of timber to be supplied.

In the matter of the limitation mentioned by Mr. Tillson, the speaker heartily concurs. It would not be reasonable to conclude that, simply because "a block might possibly wear out before it would rot out", it would be wise to use a very light treatment.

It would be necessary to use a certain quantity of oil in order to protect the blocks against bulging or "buckling". The proposition to use untreated timber is also unwise, because it has been observed that, if certain varieties of pine, if untreated, are exposed to the weather, they will be destroyed by fungi and other destructive agents in less than 4 years.

Mr. Meeker's remarks with regard to proper proportioning and mixing are extremely important. Several engineers of the Bureau of Highways, Manhattan, have endeavored for the past 2 years to restrict the use of machine mixers to those of the "batch" type. It has been noted that many "continuous" mixers furnish a mix which is often too "lean", and at other times practically devoid of stone.

If continuous mixers are not considered safe for use on such structures as the Panama Canal, the Keokuk Dam, the New York Subway and the Aqueduct work, their use should not be permitted for mixing concrete for use in foundation for pavements designed for heavy traffic.

William M. Kinney, Jun. Am. Soc. C. E.—Referring to concrete pavements in which there appears to be some pulverizing action under the bituminous carpet, the speaker has made quite a number of in- Mr. Kinney.

Mr. Kinney. vestigations of pavements of this type and, in the few cases where such condition exists, has found that the surface of the pavement, prior to the application of the bituminous carpet, was of poor quality, either because of the use of poor aggregate, or because the surface of the concrete had been frosted.

In floating concrete made from an aggregate in which there is an objectionable quantity of dirt or loam, this unsatisfactory fine material is brought to the top of the concrete and makes a surface which is of poor strength and to which another material does not readily bond. A skin on the concrete thus formed might show the pulverizing action to which Mr. Crosby has referred.

When the concrete has been frosted prior to the application of the bituminous top, practically the same result would be expected, as the thin layer loosened from the concrete by the freezing action would pulverize rapidly under travel.

When properly made concrete is covered with a bituminous carpet, and this carpet later peels off, there is usually very little material such as sand, gravel, or mortar clinging to the bituminous material.

It would be interesting to know the location of the roads to which Mr. Crosby refers, and to ascertain whether a report on the character of the aggregate and time of doing the work is available.

Mr. Goldsmith. WILLIAM GOLDSMITH, ASSOC. M. AM. SOC. C. E.—Speaking about harmonics and what Mr. Washington has said, the speaker would like to relate one little incident which happened on Second Avenue, in New York City, which brings out that point rather clearly.

From 1st to 23d Streets, on Second Avenue, experimental pavements have been laid and on one stretch, between 15th and 17th Streets, the pavement is of asphalt block. This section is in itself an experiment, different qualities of blocks being used within the area. One part of it is laid with soft blocks having a large percentage of bitumen. Other sections are laid with harder blocks.

About a year after the pavement was opened to traffic, a number of depressions and holes appeared, and the wave effect which has been mentioned was also evident. It became necessary to replace these defective parts, and in ripping up the old blocks, a curious condition was found. The asphalt blocks which were lined up perpendicular to the curb had taken a semicircular form; they had moved up on one another under the heavy traffic, so that waves were formed. Originally, the blocks were 5 in. wide, 12 in. long, and 3 in. deep. When removed their depth was found to vary from a minimum of 1 in. to a maximum of about $6\frac{1}{2}$ in. The width and length had correspondingly increased or diminished, showing that there was little if any wear of the material, but that a distortion of the blocks had taken place. This showed conclusively that the asphalt moved or crept up and worked into semicircular shape, because the blocks were soft, and not on account

of rolling. Adjoining these particular blocks were some of a harder nature which retained their shape, thickness, and alignment. Mr. Goldsmith.

Approximately every 6 ft., and perpendicular to the curb, steel plates were embedded in the concrete foundation, extending up into the asphalt block joints. The object of these plates was to prevent the creeping and distortion of the blocks, which was anticipated. The plates, however, took the same semicircular shape with the blocks, the pressure evidently being great enough to detach them from the concrete in which they were embedded.

There was no grade on that part of the street where these blocks were laid. The traffic, both light and heavy, moved in the direction of the curves, and the latter had a middle ordinate of about 7 ft., the chord being about 20 ft.

This incident seems to show that in this case the wavy effect was due to the softness of the material under heavy traffic, and not to any other cause.

There is another point, which has not been spoken of, but is of great importance. It is the factors which limit the selection of pavement to be used in the railroad area in city streets—asphalt, wood block, granite, or other material used in city streets. Many streets have car tracks in the center. The question always arises, shall the same pavement that is used in the roadway be placed within the railroad area?

According to the usual railroad laws and charter requirements, the pavement between the railroad tracks and for 2 ft. on each side of them, is maintained by the railroad company, but must be constructed in accordance with specifications prepared by the city engineer.

Investigations in New York City have shown more depressions and other defects on pavements between the tracks and near them, than at other places. The question, therefore, naturally arises, which is the best pavement to place in the railroad area? Asphalt, wood block, or granite?

The consensus of opinion in Manhattan Borough seems to be that the improved granite block is the most satisfactory. It has some disadvantages, however, on account of the ties which come in between the slot rail and track, which make it necessary to chip the granite blocks to make them fit well between these cross-ties.

The joints, in Manhattan Borough, are made with paving pitch and sand. The speaker is informed that the street railroad companies in Brooklyn have used cement grout almost entirely during the past 8 or 10 years, and very successfully.

This subject is brought up with the hope that some one will discuss it and give his experience with relation to pavements in the railroad area.

Mr. Ricker.

GEORGE A. RICKER, M. AM. SOC. C. E.—In reference to this subject of materials some curious things have happened in the New York State Highway Department, as disclosed by recent investigations, and steps are being taken to try and correct mistakes that have been made.

Roads were designed by some of the engineers who had never seen the territory in which the road was located. Specifications were applied to sections of which the engineers were totally ignorant of conditions. The speaker found places where solid rock was being excavated, and the road replacing it made of concrete, and where bituminous roads were built on blow sand foundations. At present the Division Engineers are required to send in, with their plans of roads to be improved, a report of their personal inspection after walking over them.

The Department has also instituted a survey of the State to locate available materials. One of the Assistant Engineers, who is somewhat of a geologist, and has been engaged in the Testing Bureau for some years, has been assigned to the work of examining materials in all parts of the State, and a chart is being compiled as a record of his examination.

Mr. Goodsell.

D. B. GOODSELL, ASSOC. M. AM. SOC. C. E.—The speaker cannot say much about "harmonic waves" in asphalt or bituminous pavements, but readily agrees with what Mr. Washington has said regarding Mr. Crompton's ideas as to rolling. He believes that waves such as mentioned by Mr. Crompton might occur in some hard homogeneous material, but, with one as plastic as a bituminous pavement, such waves are likely to be due to two causes: lack of cross-rolling, and lack of homogeneity of the material. Most highway engineers are familiar with the effect of cross-rolling, and the speaker believes that, if roads generally were rolled crossways, as well as longitudinally, the regular waves would not occur as frequently.

No doubt the uniformity of the material, or, in other words, the number of voids in it, has a very great influence on the evenness of wear; and the secret of obtaining a good road, other things being equal, seems to be the even distribution of voids throughout the material.

Attention is called to the measurements of wear of pitch-macadam pavement made by Mr. John Brodie, City Engineer of Liverpool, which seems to be the only instance of systematic measurement of wear under known tonnage of traffic. Some method such as he uses seems to be desirable for the determination of the relative life of the various kinds of hard pavement, including asphalt block. The speaker is not informed of any measurements which show the relative wear of asphalt-block pavements, and believes that this subject is an interesting field for investigation.

HENRY W. DURHAM, M. AM. SOC. C. E.—Reference has been made to stone block pavements in the car tracks, and to the different methods of filling joints. The speaker thinks that one important point in New York City practice was not brought out as clearly as possible. It explains very largely the difference between the practice in the Boroughs of Brooklyn and The Bronx and that adopted for Manhattan, and is mostly due to the radical difference in the types of track construction. In Manhattan an underground current-conductor with central slot between the rails is used almost entirely. In all the other Boroughs current is taken from an overhead wire, necessitating no special work in the track. The Manhattan construction requires the possibility of very frequent access to the substructure, for repairs either to the current-conductor system or the cast-iron yokes carrying the track. Therefore, it has been found more desirable to use a type of pavement which can be easily removed and promptly restored after repairs are made. Mr. Durham

The use of a cement grout as a filler gives an excellent surface between the rails, as may be seen on some of the Brooklyn and Bronx streets; but if this were used in the Manhattan track construction it would render very difficult the making of any openings for repairs to the numerous hand and manhole boxes and other sub-surface structures, and it would also be difficult to obtain proper setting of the grout filler after repairs were made. This largely explains the difference in practice.

WILLIAM DE H. WASHINGTON, ASSOC. M. AM. SOC. C. E.—Based on experience as a contractor and engineer, the speaker wishes to emphasize the importance which he attaches to traffic censuses in connection with this topic. In this matter, much may be learned from European engineers, as traffic censuses have been taken for many years in both France and England. Such censuses, and soil and geological surveys, should form an important part of the work preliminary to the design of any highway. Mr. Washington.

During its investigations in New York State, Commissioner Carlisle's Board of Consulting Engineers,* realizing the variety of local conditions obtaining in different parts of the State, recommended a "material" survey of the State, in order to make available to the Highway Department information of inestimable value if local materials were to be used economically.

As an example of proper utilization of local material might be cited the case of a section of Route 4, extending across the southern tier of counties of New York. It was found that an appropriation of $1 000 000 was available for the construction of more than 150 miles of road. In order to meet the demands of the people for a maximum mileage to be constructed under this appropriation, the engineers had designed a single-course semi-Telford road. The local material

* Report of the Board of Consulting Engineers, 1913.

Mr. Washington.

available was a soft sandstone which would rapidly disintegrate under wear and weather. Due to long hauls from railroad stations, it was found that the importation of limestone for the base and trap rock for the top would require the expenditure of a large sum of money. The Board of Consulting Engineers, therefore, decided to construct a cement concrete top, using for the mineral aggregate the local sandstone, on a base of sandstone, and to complete the pavement with a bituminous carpet.

Highway engineers require more definite knowledge relative to the life of various types of roads when subjected to varying local conditions. Massachusetts, apparently, is about the only State which has made a careful series of records covering the wearing qualities of certain classes of materials and types of roads under given traffic conditions. Every State should emulate Massachusetts and endeavor to ascertain, under their local conditions, the value of the materials which are available for road building and the service which a given type of road will give under known traffic, and careful records should be taken covering the rate of wear under traffic and the deterioration due to various climatic conditions.

Mr. Crosby has mentioned the subject of harmonic action and harmonic waves in road surfaces. This expression was evolved by that able engineer and road builder, Col. R. E. Crompton, of London, who has ascertained that roads have a tendency to creep or to form miniature hills and hollows. With certain types of traffic, he has found that the distances between the hills and hollows have a certain relative length. The speaker, in company with Col. Crompton, examined one road on which there were more than 1 000 motor busses per day, and found a series of long waves, which occurred with great regularity throughout the length of the road. The speaker believes that improper rolling of the road and the compression of the material are the main factors influencing the formation of harmonic waves. Traffic further develops the harmonic waves due primarily to the vibration of the motor cars passing over them. During the process of rolling it will be noticed that the material rises in front of the front roller and pushes ahead to a certain extent, finally ceasing to move forward, at which time the roller goes over it. If examined, the road surface will be found to have a marked swell. Col. Crompton has endeavored to solve this problem by devising a machine with three sets of rollers, instead of the ordinary tandem roller. The first and third rollers are somewhat lighter than the second one. The first roller consequently smooths and compresses the material, but does not put excessive weight on it. The second and third rollers give the additional compression necessary. It has been found possible to secure a much smoother base and surface for the road with this triple roller than with the ordinary tandem roller.

P. E. GREEN, M. AM. SOC. C. E. (by letter).—It is gratifying to find such a general agreement as to the main points among the eminent engineers who have engaged in this discussion. Naturally, there have been some sharp disagreements, but these have been rather few. Apparently, road construction and maintenance are becoming standardized, a result to be desired in many ways. Mr. Green.

Mr. Whinery seems to object mildly to the suggestion that the esthetic features of the highway should be considered, but it is believed that his objections are not well taken. It is so easy to consider them, the result is so desirable—often at little or no additional expense—that it would appear poor policy, both from a utilitarian and artistic point of view, to ignore such benefits. Too often engineers are justly charged with lack of appreciation of that very point.

It is believed also that Mr. Whinery is incorrect in his statement that the temporary and inadequate railways built in the early days resulted from the undeveloped conditions of the principles and requirements involved, rather than from financial or economic considerations. Undoubtedly, much poor engineering was perpetuated, but also much of it was of the highest order, in that it adapted its construction to its pocketbook. It is comparatively simple to build well with an unlimited pocketbook, but certainly a higher order of intelligence is often required to do well with little money.

As to the roadway diagram, Fig. 1, it would appear as if Mr. Whinery failed to use it with the discretion advocated by the writer. According to the diagram, the roadway width of the boulevard having 600 vehicles per foot of width should have been at least 80 ft., instead of 38 ft., as it actually is, and, if future traffic increase is taken into account (as it should be), the width would be increased accordingly. It cannot be maintained that the roadway width is a straight-line function of the traffic. It is believed Mr. Whinery would think better of the diagram in question if he studied it and its text a little more carefully.

Mr. Tillson encountered the same obstacle that the writer found; that is, the difficulty in describing "new matter", but he got around it more gracefully. However, the writer disclaims its authorship; it was simply one of the Society's rules governing this discussion.

Mr. Tillson further calls attention to the late practice of the City of Paris in treating paving blocks with oil under pressure to the amount of 10 or 12 lb. per cu. ft. This is largely for the purpose of preventing the blocks from swelling. The writer, however, believes that swelling will not be prevented in that manner. At Longview, Tex., in September, 1913, after a hot dry summer, it was found that blocks treated with true creosote oil in 1911 absorbed nearly 50% of their volume of water and had lost nearly 50% of the contained oil.*

*Engineering News, December 4th, 1913.

Mr. Green. It is believed that there is only one way to treat wood blocks permanently so that they will not absorb a great deal of water, and that is by traffic.

The writer thoroughly agrees with Mr. Meeker and Mr. Washington that the so-called "harmonic" waves are caused more by poor workmanship and materials than anything else. On Diversey Boulevard, Chicago, paved with bituminous concrete in 1911, at a section having a mixed traffic of about 5 000 vehicles per day, evenly divided as to direction, one side has pushed into these waves so that it has become exceedingly rough. The other side is quite smooth. This condition is apparent for only about 2 000 ft. The rest of the highway (several miles) is in perfect condition. It is very evident that defective material or workmanship or both have been the cause of this.

(3) EQUIPMENT AND METHODS FOR MAINTAINING BITUMINOUS SURFACES AND BITUMINOUS PAVEMENTS.

By Messrs. William R. Farrington, H. B. Pullar, Arthur H. Blanchard, W. H. Fulweiler, James H. Sturdevant, Herbert Spencer, T. Hugh Boorman, Philip P. Sharples, William de H. Washington, A. F. Masury, and William R. Farrington.

William R. Farrington, M. Am. Soc. C. E. (by letter).—The maintenance of bituminous surfaces and bituminous pavements is divided naturally into three stages. The first includes what is usually classified as ordinary maintenance, such as patching, covering, etc. The second covers re-treatment of the surfaces and the renewal of the seal coats on the pavements; and the third includes the re-shaping or re-surfacing of roadways carrying bituminous surfaces, and the re-surfacing or reconstruction of the pavements. Mr. Farrington.

The writer will discuss the work with which he is familiar, on roads outside the cities and large centers, and will consider the equipment needed and the methods to be followed in maintaining bituminous surfaces consisting of hot or cold oil, or tar, and sand, gravel, or chips; also, bituminous pavements composed of broken stone and tar, oil or asphalt; or of sand and oil.

It is not intended to take up the cost of maintenance, for though this cost might be regarded as a criterion of suitable equipment and proper methods, it probably varies so much in different localities that, after all, a comparison of costs is not of much value in a general discussion, unless the conditions are well understood and the prices paid in any locality are known.

It is assumed that maintenance work will usually be handled by a department with its own gangs and equipment, rather than by contract, except possibly in case of extensive re-surfacing.

Patching.—For patching, only a small gang is needed, and the equipment is simple. There should be one or two portable melting kettles, a small portable tool-house, and one or two small tents for the laborers, unless a van is provided.

The methods to be followed in patching the different kinds of surfaces and pavements are much the same. Generally, one should use the same kind of bituminous material for patching as was used in forming the surface, and ordinarily it is not advisable to select a much heavier grade, except with stone, slag, or chips, for filling depressions.

In the writer's opinion, only the lightest, or so-called dust-laying, oils or tars, should be used without the addition of some grit; and, even with these, a light covering is probably economical. If applied

Mr. Farrington. without covering, however, patching is usually not advisable, as the cost of an additional application is small.

In patching an ordinary oil or tar surface, one should first attend to the depressions. Slight depressions can be filled with oil, or tar, and sand, gravel, or chips; but if they are deep, stone or slag should be added. The depressions should be cleaned, and the edges cut out if necessary, in order to allow a thorough bonding of the new and old material.

Although the best results are obtained by mixing the bituminous material and sand, or stone, etc., before applying, satisfactory results may be obtained without mixing, if the work is done carefully. In filling slight depressions, where the materials are not mixed before applying, a thin coating of oil or tar is spread over the bottom and covered with sand, gravel, or chips. If stone or slag is used with oil for filling deep depressions, the stone or slag is spread over the oil with which the bottom has been coated, then more oil is applied and covered with grit. If tar is used, the methods are similar, but more bituminous material is required.

After filling the depressions, any bare places, or places where the bituminous surface is practically worn through, should be coated, the surface first being cleaned and grit being spread over the oil or tar. In no case should much more than ½ gal. of bituminous material per square yard be used for coating the surface, as any surplus oil or tar will result eventually in the rolling of the surface and the formation of bunches.

Most of the so-called cold tars may be applied without heating, as may the lighter grades of the so-called cold oils, or road oils; but the heavier grades must be heated in order to give the best results in patching.

A much heavier oil or tar than that used in the surface is inadvisable for patching, yet a somewhat lighter grade may be used. Excellent results are obtained by patching a hot oil or tar blanket with the heaviest cold oil or tar, and with these there is much less chance for the formation of bunches than with the heavier materials.

If a pavement composed of stone and bituminous material is constructed properly, not much in the way of maintenance should be required for some time, but depressions will develop eventually, and some patching will be needed. In patching bituminous pavements, including those composed of sand and oil, the methods are similar to those for a bituminous surface, except that, with the sand and oil pavements, deep depressions should be filled with heavy oil and sand, properly heated and mixed before applying; or, if the heavy oil is not available, or there are no facilities for heating the sand, an oil similar to that used for surface work will give good results with slag or cinders, without mixing.

Covering.—On any bituminous surface, and especially one in the formation of which oil is used, more or less covering with sand, gravel, or chips will be required, either to take up free oil or tar, which will work to the surface for some time after it is applied, or later to prevent the picking up of the surface, which may soften under certain conditions. Covering is also partly a remedy for the slippery condition incidental to any bituminous surface during cold weather. Mr. Farrington.

Re-Oiling or Re-Tarviating.—For re-treating a bituminous surface, practically the same equipment is required and the same methods should be followed as for the original treatment, except that usually less bituminous material is needed.

The life of a bituminous surface, or the length of time elapsing before re-treatment is necessary, will depend on local conditions, and the amount and kind of travel, etc. Where conditions are favorable for such treatment, however, and the travel, especially the horse-drawn travel, is not too heavy, a bituminous surface consisting of grit and about ½ gal. per sq. yd. of the heaviest hot oil suitable for blanket work should last from 3 to 5 years without re-treatment, if patched occasionally and otherwise maintained properly. In re-treating such a surface, not more than ¼ gal. of oil per sq. yd. should be used, unless practically all the original surface is gone; and it is not economical to delay re-treatment until this point is reached, as without the protection of the bituminous surface, the road metal will tend to ravel and the roadway to disintegrate.

A bituminous surface formed by the application of from ⅛ to ¼ gal. of light oil per sq. yd., with a small quantity of covering, will last at the best not more than one season, and it will probably be necessary to cover such a surface quite heavily in the fall in order to carry it through the winter and until another application is made the following year. Where from ⅜ to ½ gal. per sq. yd. of the heaviest cold oil is used with grit, re-treatment with about the same quantity of oil will usually be necessary the second year; but, after that, the application of from ⅛ to ⅓ gal. per sq. yd. per season should be sufficient, and an application may not be necessary each year, although, if omitted, the patching will naturally be increased.

If refined coal-gas tar is used, about ½ gal. per sq. yd. is required during the first and second seasons. After that, with cold tar, a ¼ to ⅓-gal. application will usually be necessary each season, although the yearly application may be omitted occasionally. With hot tar, ordinarily a ¼ to ½-gal. application should be made every second year. Where unrefined water-gas tar is applied, two ⅜-gal. applications are advisable each year; but, if only one ½-gal. application is made, a considerable quantity of patching will be required.

As to equipment, for handling the lighter grades of bituminous materials, one or two distributor carts and a pump for transferring

Mr. Farrington. these materials from the tank-cars are required, and there should be a tool-house, also tents for the laborers, unless vans are provided.

For handling the heaviest grades of oil or tar used for surface work, the following plant is needed:

One 10 or 12-ton steam roller,
One portable heating boiler,
One steam pump,
Two steel tank-wagons,
One pressure distributor (if mounted separately, or two if attached to the tank wagons),
One or two watering carts,
One 2-horse sweeper,
One portable tool-house, and
Two large tents or vans for laborers.

As to distributors, for handling the road oils or the corresponding grades of tars, a gravity machine may be used with good results, if the bituminous material is broomed after it is applied; but the cost of distribution will be less and the results more satisfactory if a pressure distributor is used. For this work, a distributor cart with a pump mounted on a frame back of the tank, and connected by chain with a large sprocket bolted to one of the rear wheels is most satisfactory. Under favorable conditions, good results have been obtained with gravity machines in distributing even the heaviest grades of bituminous materials for surface work; but the chances of failure, if the conditions are not favorable, are such that they should always be applied by pressure distributors if possible. The distributor just described, or one with a gas-engine pump mounted either on the tank-wagon or on separate gear, may be used, but one operated by steam will give the best results. One advantage of the steam distributor is the possibility of cleaning out the pipes and nozzles by blowing steam through them. Of the steam distributors, there seems to be little choice between the one using the direct pressure of steam in the tank for distributing and the pump type, each having its advantages.

To facilitate covering, the sand or other material should be piled in advance on the side of the road.

Before commencing to distribute the bituminous material, the surface should be cleaned with a horse sweeper, and with hand-brooms, if necessary. Any depressions should be filled and the surface patched as needed.

Where the heavier grades of oil or tar are used, it is well to water the old surface before distributing the new bituminous material, but watering is not necessary with the lighter grades unless the surface is unusually dry.

Mr. Farrington.

It is advisable to cover the bituminous material lightly at first, applying more grit as needed, for though some bituminous material will take up only enough grit to form a good mixture, without reference to the quantity put on, in other cases, if there is surplus covering, the oil or tar will continue to work up until the mixture is weakened to such an extent that the best results cannot be obtained.

Seal Coats.—Although a seal coat or wearing coat may not in all cases be necessary on pavements composed of broken stone and bituminous material, the writer believes that it is economical and should usually be applied, either in connection with construction, or later, except possibly where asphalt is used in the pavement. The equipment and methods for applying or renewing such coats are similar to those for the re-treatment of the surface; but, unless asphalt is used, the bituminous material for the seal coat should be of a lighter grade than that incorporated in the pavement. Oil may be used with good results on a tar pavement, and is advantageous, in some cases, in that it is less slippery than tar and also acts to some extent as a dust absorbent.

Re-Shaping or Re-Surfacing.—Although a surface consisting of grit and suitable bituminous material will carry ordinary travel and relieve the road metal of much of the wear, it alone is not a permanent form of treatment. As a result of the gradual wear of the road metal, and because repeated treatment and patching with bituminous material will cause the formation of bunches and an uneven surface, re-shaping or re-surfacing of the roadway will be necessary, eventually.

When this stage is reached it is always well to consider a more permanent form of pavement; for it is an established fact that a bituminous surface will not carry a large amount of horse-drawn travel, and will fail under certain conditions which the writer believes are not as yet clearly understood; and although a bituminous surface may have been satisfactory in a certain locality in the past, the conditions may change or the travel increase largely.

It is assumed, however, that there will not always be money available for the construction of a bituminous pavement, in which case the alternative is the re-shaping or re-surfacing of the roadway and the application of another bituminous surface.

The equipment needed for re-shaping or re-surfacing work, to be done in connection with re-oiling or re-tarviating, is the same as that for re-treatment, except that two steam rollers will be needed, also a straight-tooth and a spring-tooth harrow.

If stone is to be crushed, there should be a portable crushing outfit.

Mr. Farrington.

In re-shaping or re-surfacing a roadway with a bituminous surface, the old surface should first be thoroughly broken up with the roller picks or a scarifier and removed with stone forks.

The loosening of the road metal, partly accomplished by the roller picks or scarifier used in breaking up the surface, should be completed with a straight-tooth harrow, the use of which also tends to shake the worn out material to the bottom. This should be followed by a spring-tooth harrow, which, because of the curved teeth, will bring the stone or larger particles of road metal to the top. The surface should be shaped with rakes, any new stone or other road metal needed should be put on, and the road watered and thoroughly rolled. If any considerable quantity of new stone, etc., is used, the voids should be partly filled with sand, stone screenings, or chips; but, if only a small quantity of new road metal is put on, there will usually be sufficient fine material in the road to bond it.

Either a macadam binder or the heaviest road oil may be used, but, ordinarily, the former will give the best results. About ½ gal. of oil per sq. yd. should be used in two applications, with a covering of grit after each.

Ordinarily, it is not advisable to use tar for such re-shaping or re-surfacing work, unless a bituminous pavement is to be formed by penetration. In this case, usually more new stone or other road metal is needed, and the voids for about 2 in. below the surface should not be filled with fine material. For grouting, 1¼ gal. of heavy tar should be used, and a seal coat should be formed by the application of about ½ gal. of a lighter tar with grit; or, if preferred, asphalt may be used instead of tar.

The foregoing description of maintenance work applies to a bituminous surface on a roadway of broken stone, slag, or screened gravel, rather than to such a surface on unscreened gravel; but though the methods for handling a surface on the latter form of roadway are somewhat different, mention can only be made of the fact that although, for the first application, one should use a fairly light grade of bituminous material which will penetrate somewhat below the road surface and thus form a real bond between the bituminous material and the gravel, heavier grades of oil or tar may be used, with good results, for later treatment.

Re-Surfacing of Bituminous Pavement.—In a discussion on maintenance, it does not seem advisable to devote much space to the re-surfacing of bituminous pavements, because such work is probably better classified as reconstruction than maintenance, but attention is called to the fact that the life of pavements composed of broken stone and bituminous material is as yet problematical on roads outside of cities

and large centers, because such pavements have been in use, under these conditions, only a few years. Mr. Farrington.

Storage and Repairs.—Incidental to a consideration of equipment is the question of proper care and up-keep of the outfits. There should be at headquarters a suitable storage place, with sufficient equipment and proper facilities for making ordinary repairs. It is not only advantageous to have the repairs made by or under the supervision of those responsible for the apparatus during the working season, but, by having the work done by men from the regular force, it is possible to keep the most competent men busy during the winter.

If the amount of work done warrants the employment of a store-keeper, there should be kept at all times, at the storage place, a sufficient stock of tools, etc., to supply the needs of all the gangs.

Reports and Accounts.—It is not possible, in the time at the writer's disposal, to consider in detail the question of the reports, records, and accounts which should be kept on maintenance work. The writer believes, however, that although a consideration of cost is not of especial value in a discussion covering work in widely separated localities, a comparison of the cost of work in different localities in one section is valuable, and is necessary in order to determine what equipment and methods are most economical, and also whether the work is handled efficiently.

Daily reports giving details of the labor cost should be submitted by the foreman, and a record of these, as well as of bills for materials, supplies, etc., should be kept in such a way that the cost of the different kinds of work can be easily ascertained at any time.

H. B. Pullar, Assoc. Am. Soc. C. E. (by letter).—The methods of maintaining bituminous surfaces and pavements are unquestionably of great importance. Mr. Farrington has covered very fully the matter of maintaining bituminous surfaces and bituminous roads constructed by the penetration method. He has stated that he does not consider it advisable to devote much time to the re-surfacing of bituminous pavements. Although it may not be advisable to devote much time to the total re-surfacing of bituminous pavements, much more might be said regarding their maintenance and repair. The method and equipment necessary for repairing sheet-asphalt and asphaltic-concrete pavements in small towns and cities have been of great importance, and it is on account of the fear of the cost and trouble in making satisfactory repairs that many small towns have given preference to brick, block, and other types in place of bituminous pavements. If proper care is taken and proper equipment is secured, there is no more trouble, nor any greater expense, in repairing bituminous pavements satisfactorily than with Mr. Pullar.

Mr. Pullar. any other kind. Methods and equipment necessary for repairing these pavements will be discussed later.

It has only been within recent years that much consideration has been given to the repairing of sheet-asphalt pavements. Until recently, the only thing which could be done was to cut out the bad part of the pavement and replace it with new. On streets where the total repairs are more than 50 or 60% of the pavement, this is still the best method. Where this plan is used, all the bad material should be cut out, the edges of the old pavement should be carefully painted with the pure bituminous cement, and then the new material should be put in by a method similar to that used in constructing the pavement. As Mr. Farrington states, an effort should be made to make the repairs with materials of the same class as those used in the original pavement. In the larger cities, or wherever it is possible, the most economical way to obtain the mixture is from some paving contractor. In smaller cities it is advisable to obtain one of the small mixing plants which are on the market and now available. The "Rapid" mixer has proved to be of exceptional value for this work, and is now used by a number of cities in the Middle West for repairing both sheet-asphalt and asphaltic-concrete pavements.

For sheet-asphalt pavements, in which only small depressions or waves appear, the most economical and probably the best method is to burn off about ½ in. of the top and make the repairs by what is known as a "skin patch". This method has been used very successfully in various parts of the country, and there are a number of excellent burners on the market. The burning can be done at about the same expense as would be involved in ripping up the pavement, and in this case it is only necessary to put on a patch about ½ in. thick, instead of cutting out and replacing the entire thickness of the pavement.

The "Lutz" heater method for repairing is based on the same principle—burning off a small quantity of the top surface and replacing it with a "skin patch". The surface is heated by a hot-air blast, and claims are made that no injury is done to the pavement, with the exception of the upper ½ in. which is removed. This heater has been used successfully by a number of the larger cities for re-surfacing work, and it is possible to repair about 500 sq. yd. per day with one machine. Care and experience, however, are necessary to repair or re-surface pavements properly by burning. It should not be attempted by anybody without some knowledge of the proper method. It is not a method which can be recommended for general use in small cities or towns, or on public highways.

During the past few years many attempts have been made to re-melt and use the old material for repair work, but, up to the present time, no successful method has been developed. The Noyes crushing machine has been used successfully for pulverizing the old material,

but the reheating has proved a difficult matter, and thus far has not met with success. The material, after being pulverized by the Noyes crusher, can be used to a considerable extent in a binder mixture, and this idea is being adopted in a number of places. Some of the small mixers claim to be able to re-melt old material satisfactorily, but in nearly every instance considerable burning takes place. **Mr. Pullar.**

About 2 years ago the writer made a new experiment in repairing sheet-asphalt pavements. The section of pavement to be repaired was very badly cracked, and contained three holes, each about 1 sq. yd. in area. The holes were first cleaned out and the edges trimmed. Stone, ranging from ¼ to 1 in. in diameter, was then placed in each hole and tamped, and to this there was applied about 1 gal. per sq. yd. of a heavy asphalt binder. Chips were then spread over the surface and thoroughly tamped into the voids. Then a squeegee or seal coat was applied over the entire surface, including the stone patches. A recent inspection of this small strip of pavement showed that it was still in excellent condition, so that, without question, this method has already prolonged the life of that particular piece of pavement 2 years, and from indications, will continue to prolong its life another 2 or 3 years. On account of the success of this experiment this method has been used for repairing sheet-asphalt pavement in two or three other small cities in various parts of the country, and up to the present time, reports have been very favorable. This method could be used very satisfactorily in small towns and villages, and would be much more economical than the use of brick or cement for repairing bituminous surfaces.

In repairing asphaltic-concrete pavements of various types, the best method is to use a small mixing plant, such as the "Rapid" mixer, or other small mixer of similar construction. The nominal cost of these machines and the ease with which repairs can be made, will probably result in their more general use for repair and maintenance work.

There are two principal causes for trouble with bituminous surfaces consisting of the superficial treatment by use of tar, oil, or asphalt. One is the fact that, in a number of instances, too much bituminous material has been used. This produces a pavement which is very apt to rut or become wavy, and it is also very disagreeable to the eye. The other source of trouble is due to lack of sufficient bituminous material, or too great a length of time intervening between treatments. If the surface is carefully inspected at frequent intervals, and treated at the proper time and with the proper quantity of material, the road or pavement can be kept in excellent condition at a small cost.

The writer agrees with Mr. Farrington in the statement that it is not advisable at present to discuss the cost of maintenance, as this

Mr. Pullar. varies considerably in different parts of the country, and, to a great extent, depends on local conditions. Cost data would be valuable even when work in different localities is considered, provided there was a standard system for giving cost, and one which would take care of different units. The total cost per square yard is of little importance unless unit costs are given. A standardization of a system for giving cost data, in both paving and road construction, and in paving and road maintenance, is greatly needed, and much valuable work could be done by a committee of engineers appointed for this purpose.

The question of equipment and methods for maintaining bituminous surfaces and bituminous pavements, is a very broad one, and it is impossible to cover it fully in one discussion, but it is hoped that further experience may be cited on this same topic, especially in reference to a cost system and to the best methods for use in small towns and villages.

Mr. Blanchard. ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—Mr. Farrington properly considers the work of maintaining bituminous surfaces and bituminous pavements under three heads: first, routine maintenance, being the continuous repair of small areas of a surface or pavement; second, re-applications of bituminous materials over practically the whole surface; third, reconstruction of the wearing course.

The second and third classes of maintenance may be accomplished by day labor or contract, or a combination thereof, the problems involved being usually identical with those of primary construction. The relative advantages of the general methods referred to and the plant equipment required have formed the subject-matter of several previous discussions before this Society.* The speaker will confine his discussion to the field of maintenance work first mentioned above.

In considering the equipment required to accomplish continuous maintenance of bituminous surfaces and bituminous pavements, it seems desirable to cite certain factors and principles which appear to be essentials of economical and satisfactory maintenance.

1.—Small failures of the wearing course, and the wearing away of the bituminous surface on comparatively small areas should be repaired immediately. Otherwise the rate of disintegration will be materially increased, the annual maintenance charge will be greater than necessary, and the surface will be unsatisfactory to the users of the highway.

2.—Continuous maintenance should be conducted so that the highway may be at once opened to traffic without injury resulting to the repaired sections. For the repair of potholes, ruts, and other holes or depressions, this implies the use of bituminous materials of much lower penetration than have ordinarily been used in many cases. If

* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 25; Vol. LXXV, p. 548; and Vol. LXXVII, p. 171.

practicable, the same type and grade of bituminous material should be used for the surface application as was placed in the original construction. Mr. Blanchard.

3.—In order to obtain satisfactory results, the labor should be efficient and should be under the supervision of an engineer experienced in the construction and maintenance of all types of highways, and with all kinds of bituminous materials used in the area under his jurisdiction.

4.—The intimate relationship between the methods and materials used in construction and the character of the maintenance, in the case of bituminous surfaces and bituminous pavements, is such that, to ensure success, the construction and maintenance of highways in a unit of area, or a certain mileage, should be under the supervision of one engineer.

To accomplish continuous maintenance economically and efficiently under the foregoing conditions, the flying squadron, operated under the direction of an engineer in charge of construction and maintenance of highways in the area covered, appears to be the logical solution. The equipment of such a squadron will naturally depend on local conditions, such as the mileage of highways to be maintained, their relative location, the types of surfaces and pavements, the kinds of bituminous materials used, the mileage under guaranty, etc.

As an example of a definite problem in continuous maintenance, take a county or a division of a State where a considerable mileage of bituminous surfaces and bituminous pavements has been constructed. Granted a well-maintained system, the work to be accomplished by a flying squadron would consist, first, of routine repairs, including filling all holes, ruts, and other depressions with bituminous concrete, using a type of aggregate and bituminous material suitable to each case, and, second, applying bituminous materials to all areas which gave indication of being in such condition that they should be re-treated at once rather than wait until the whole surface required another application. It will be practicable in many cases for the flying squadron to repaint guard-rails and perform other routine repair work. Under such conditions, it is believed that the use of a motor truck properly equipped will prove most satisfactory. The suggestion to use a motor truck in repair work on highways is not novel, trucks having been recommended and used for this purpose for several years.

The equipment to be carried by the motor truck for the special repair work outlined should include the following machinery and tools: a rotary heater and pug mill mixer; two heating tanks; a surface heater; storage tanks and barrels for bituminous materials of different types and grades; storage capacity for small tools such as brushes, squeegees, tampers, cutters, pouring cans, irons, shovels, picks, and hoes; and storage capacity for paints for guard-rails, mineral aggregates, and road metal of several sizes. Although a storage capacity

Mr. Blanchard. for mineral aggregates and road metal is called for in this equipment, it should be noted that only a few cubic feet of these materials will be carried, because the maintenance work performed will be confined to the repair of small patches. Usually, it will be practicable to reload a truck each day. The fixtures for the equipment described should be arranged so as to be readily removed, thus allowing the truck to be utilized for general hauling purposes. The equipment recommended should not be confounded with the elaborate equipment designed to cover all phases of repair work recommended by some engineers. The speaker believes there is danger in overloading a truck with accessories and thus rendering its operation uneconomical.

The equipment recommended herein could be carried by a 5-ton truck, capable of traveling 12 miles per hour, and having a gasoline storage capacity for a run of 100 miles. A clear body of 7 by 15 ft. would be available for the installation of the equipment. The cost of operation during an 8-hour day would vary from $10 to $20, covering wages of chauffeur, rent of garage, interest on first cost, maintenance charges, depreciation, insurance, tires, gasoline, oil, and grease. The speaker is indebted to Mr. Alfred F. Masury, Service Manager, The International Motor Company, for details relative to truck operation.

Mr. Fulweiler. W. H. Fulweiler, Assoc. M. Am. Soc. C. E. (by letter).—It is not economical to apply a bituminous material without some covering, as the greatest value of the treatment lies in the added wear given to the surface of the road by the mineral aggregate used as a covering, which the bituminous material merely serves to protect and retain in position. It is always economical, especially in the re-treatment of bituminous surfaces, to repair the surface before applying the bituminous treatment.

In patching, it is very important to see that the edge of the patch is cut back slightly, so that it not only forms a square edge for the new material, but that the material on the edge of the patch, which is invariably partly disintegrated by the action of the water, is removed.

The writer would like to inquire whether Mr. Farrington has any explanation for his statement that "Where tar is used in patching, more of it is required than with other materials."

As to the quantity of material to be applied on the surface, the writer's experience indicates that should it be desired to apply ½ gal. as a re-treatment on a bituminous surface, the best results would be obtained by applying not more than ¼ gal. at a time—with just sufficient covering to enable the wagon or motor truck to pass over the surface—then applying the remainder of the material and adding the stone used for covering. Where such a large quantity of material (½ gal.) is used in re-treatment work, it is advantageous to use stone

of a larger size, in fact the writer's best results have been obtained with a size commercially known as "¾-in." Mr. Fulweiler.

The writer agrees with Mr. Farrington regarding the excellent results that may be obtained in patching with bituminous materials which are prepared so that they may be applied without heating, as this removes one of the serious difficulties in making a large number of small patches. Furthermore, such materials may be applied at almost any season of the year, and are much less affected by the presence of dust and moisture in the road surface.

In this day of definition and striving to attain accuracy in the use of technical phrases, the writer would like to inquire whether Mr. Farrington believes it to be good form to use a proprietary trade name in the way he has used the terms "Tarviating" and "Re-tarviating"?

In reference to the use of tar in re-treatments, the writer's experience has been that where the road is in good condition and an initial application of ½ gal. per sq. yd. is made, that under favorable conditions, patching is all that is necessary during the second season, and about ⅓ gal. per sq. yd. for the third season, followed by ¼ gal.; although, under many conditions, ⅓ gal. every other season seems to be sufficient.

The writer does not agree with Mr. Farrington in reference to applying the covering to the bituminous material lightly at first. After a little experience it is very easy to estimate the proper quantity, and better results are obtained by putting on this correct quantity immediately. This saves trouble, annoyance to traffic, and prevents the formation of spots in the road, where, owing to the small quantity of covering, the road surface will pick up, especially under heavy steel-tired traffic. It is but fair to state that the writer is drawing purely on his own experience, which has been confined practically to a single type of material.

Slipperiness frequently results from the method of putting on the stone covering, as suggested by Mr. Farrington, as the application in small quantities tends to keep an excessive quantity of bitumen in the top of the road.

In connection with the description of the organization for surface treatment and re-treatment, the writer would be very glad to have an expression of opinion from Mr. Farrington as to the number of square yards per day treated by such organizations, assuming an average haul of 3 miles from the railroad station.

Mr. Farrington mentions a macadam binder, or the heaviest road oil to be used in re-shaping the surface of an old road. By macadam binder, does he mean a particular grade of bituminous material, or the usual stone screenings well watered and rolled?

In reference to maintenance work on screened gravel, the writer is in entire agreement with Mr. Farrington, that a fairly light grade

Mr. Fulweiler.

of bituminous material should be used on the first application, followed by a heavier grade, but finds it best to apply the heavier as soon as the light material has been absorbed into the surface. This same method has been used successfully on oyster-shell roads.

In re-surfacing old bituminous pavements, the writer's instinct would always be to advise scarifying, but the results of some work done during the past season would seem to indicate that, with care, it is possible to secure very good results by using crusher-run stone, free from dust, that will pass a 1½-in. screen, applying this as a covering over about ½ gal. of material, rolling thoroughly, applying about ⅓ gal. and covering with stone chips.

In the organization for re-surfacing, where a considerable amount of this work is to be done, the writer would like to call attention to a plan by which a very large amount of such work has been done on the State roads in Maryland during the past several years. Under this plan the State furnishes and applies the necessary stone covering, purchasing the stone and having it delivered in piles along the roadside in the early spring, when the price of teams is quite low. From a schedule prepared by the State engineers, showing the location of the roads to be treated, a price is agreed on, and a contract drawn up whereby the manufacturer of the material agrees, for a unit price per square yard, to sweep surfaces which have been previously treated with bituminous material, and also surfaces which have not been thus treated, and a unit price per gallon for applying the material, at the approximate rate shown on the schedule. In this contract for the past season, it was specified that all work should be completed before July 31st, with a penalty clause of $25 per day for every day that this time was exceeded. There was a further clause in which the company agreed to pay the State $25 per day for every day that it was delayed through breakdowns or the failure of material to arrive, and the State agreed to pay the company the same amount for every day that it delayed the application of the material through inability to cover it. This contract involved some thirty-four pieces of road in nine counties of the eastern shore of Maryland, amounting to about 118 miles, and containing 976 071 sq. yd. On this was applied 342 785 gal. of material, at a rate varying from ⅓ to ½ gal. per sq. yd., and averaging 0.366 gal. per sq. yd. This was covered with stone chips averaging 12 lb. per sq. yd.

Two motor trucks were used in this work, and they arrived about May 26th, although, owing to bad weather and some delay in organizing the gangs, work was not commenced until May 28th. One truck finished practically half of the work on July 18th, and the other finished on July 30th. The State maintained two gangs, of 12 men each, each under an inspector, for applying the stone chips, the company sweeping the roads and furnishing and applying the binder.

Mr. Fulweiler.

Practically this same form of contract had been in force during 1912, but during that year the work was not laid out according to schedule, and the trucks frequently had to make long jumps between the different pieces of work. This caused considerable loss of time, which was furthermore augmented by difficulty in securing proper labor. As the result of the experience of 1912, the work was laid out more systematically in 1913, and men composing the covering gang in one case were held throughout the season. The effect of having experienced men for covering is well brought out in Table 1 which summarizes the operations of the two years.

TABLE 1.—SUMMARY OF OPERATIONS FOR 1912 AND 1913.

	1912.	1913.
Sunday and holidays	21	20
Repairs and delays	22	6
Rain	5	9
Transit	34	18
Applying material	57	72
Total elapsed days	138	125
Percentage of days worked	66%	72%
Percentage of days material applied	41.5%	57.5%
Average No. of gallons applied per day	2 860	4 760
Average No. of square yards treated per day	8 230	13 550

The time is given in truck-days.

In 1913 the decrease in time lost in transit, repairs, and delays, is quite striking, as is the average increase in the quantity applied per day.

The writer believes that, in the long run, this plan, where there is a considerable mileage to be treated, is probably the most efficient and economical method of handling this maintenance work. One point, which is of considerable importance, is that, contrary to the case of an ordinary contractor, the company is vitally interested in securing the very best possible results from the use of its material, and therefore the inspector's duties are very much lightened, as there is no incentive to skimp the sweeping or other details in the application and then blame the quality of the material for the poor results which would inevitably ensue.

Mr. Sturdevant.

JAMES H. STURDEVANT, ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Farrington has well covered the subject of maintenance, without which any road system becomes an abomination instead of an improvement. Many times the writer has listened to tourists recounting a day's experience on the roads. They have expressed great indignation concerning some rough spots, but have not mentioned the many miles of smooth, easy-riding roads over which they have passed.

There are a few minor points to which the writer would like to call attention. These may seem to be trivial, but, if attended to, will result in ultimate satisfaction to the general public.

Mr. Sturdevant.

Patching.—In order to keep the surface of a road in smooth, proper condition, it should be under constant surveillance, and receive constant attention. As the surface becomes worn in places, and holes and ruts develop, they should be repaired at once, for if neglected they rapidly extend in size and depth under the action of each traveling vehicle. On bituminous surfaces, the best results are obtained, as stated by Mr. Farrington, if the asphaltic materials are first mixed with stone, gravel, or other suitable ingredients. This mixing can be done at any time, and in sufficient quantities to last for several months, for the weather will not cause the mixed material to deteriorate. The writer has used material which had been mixed during the previous summer and piled by the roadside, and has obtained as good results as with fresh materials.

In all patching work, the first requisite is to see that the surface to be repaired is well cleaned before new material is placed; also, in repairing holes and ruts, care must be taken not to use too much asphaltic material, and it is well not to have the new patch flush with the surface, for, under the action of the sun, the asphalt, whether light or heavy, will come to the surface. Then the patch will become sticky and is likely to be pulled off by heavy slow-going vehicles. To prevent this, more stone and fine material must be used as covering, and, if too much was used originally, a hump will develop which is as disagreeable as the original rut or hole. This is not as likely to occur when the material has been mixed before placing, as when the asphalt is placed first and then covered. Where the patching has been properly done—judging from the writer's experience and observation—it will last long after the original oil and surface have disappeared. This is true as to either light or heavy oils, or even asphaltic binders.

Re-oiling.—For re-treating bituminous pavements, the quantity of material to be used varies with individual roads. The original treatment should be in sufficient quantities to bind the road thoroughly and hold the stone in place, to prevent raveling; subsequent applications are for the purpose of providing a mat or cushion. For this purpose, the lighter oils are generally used, though excellent results, well worth the extra cost, are obtained with heavy binders. Considering, however, the lighter grades of oil, it is apparent that if used in excess they are not heavy enough to sustain the weight of traffic, and a rutted condition is sure to result. To avoid this, the application of oil should be uniform and of just sufficient depth to cover the surface, with no free oil remaining. After being covered, the surface should be watched closely, especially in warm weather, and as soon as the oil appears through the surface, more material should be added. In general with all bituminous surfaces, satisfactory results can only be obtained by keeping close watch of the road and repairing all defects as rapidly as they appear.

Herbert Spencer, Assoc. M. Am. Soc. C. E.—Any discussion of a subject such as the equipment and methods for maintaining bituminous surfaces and bituminous pavements must, of necessity, be limited by the length of time such surfaces have been in existence, and should properly include: (1) classification of the surfaces and pavements required to be maintained; (2) methods and material used in their construction, together with the causes responsible for their failure; and (3) equipment and methods required for their maintenance. The subject being of such importance, and having such possibilities of detail, the speaker is of the opinion that a summary of these headings is all that should properly be attempted in a limited discussion of this nature. It must also be remembered that bituminous surfaces and pavements of various kinds having a large mileage, have ended their period of usefulness, and their maintenance now becomes a matter of first importance and requires the development of machinery and methods for their economical up-keep. These can come only from actual experience, and the lines along which such development should follow is all that can be attempted at present. Mr. Spencer.

During the past 5 years many types of bituminous surfaces and pavements have been placed on the public streets and highways. Some of these have attained sufficient merit in the public estimation to warrant their continuation, and others have shown such faults in their original design and construction that their further development was not considered economical or wise. Without doubt, new types of bituminous surfaces and pavements will be exploited with the growth of highway work, but it is felt that these cannot deviate very much from the existing types, which are defined by the Special Committee on Bituminous Materials for Road Construction as consisting of superficial coats of bituminous material with or without the addition of stone or slag chips, gravel, sand, or material of similar character. This definition may include: (*a*) treatment of gravel or stone roads with cold bituminous material; (*b*) treatment with hot bituminous material; or (*c*) treatment of a Portland cement concrete base with either hot or cold bituminous material.

The Special Committee on Bituminous Materials for Road Construction defines a bituminous pavement as "One composed of stone. gravel, sand, shell, or slag, or combinations thereof, and bituminous materials incorporated together by mixing methods."* This definition may include: bituminous concrete, laid either hot or cold; bituminous macadam (penetration method); bituminous mortars, consisting of bituminous cement and fine material aggregate, such as rock asphalt, and laid either hot or cold; sheet-asphalt, asphalt block, and similar pavements.

The maintenance of bituminous surfaces and bituminous pavements is measured largely by the methods and materials used in their original

Mr. Spencer.

construction. Although it is not considered necessary to enter into a discussion here of the specifications for any of the types just mentioned, the generally accepted limitations of methods of construction may be summarized as an index to the extent of maintenance.

For bituminous surfaces, the thorough sweeping of the base is of great importance for applications of hot bituminous materials, and should be resorted to for cold applications wherever an extremely dusty condition of the surface is found. The bituminous materials used range from asphalt oils and tars to the heavier products, of like characteristics, requiring heat for their application. Of importance is the selection of the cover. For the hot applications, coarse sand or grits, preferably washed, are commonly used for automobile traffic; but, for steep grades and horse-drawn traffic, stone screenings are better. The items, therefore, influencing the success or failure of bituminous surfaces include the preparation of the base, the selection of the bituminous material, the selection of the cover, and, of equal importance, the machinery to be used in the application of the bituminous material. The treatment of cement concrete surfaces with bituminous material is of too recent origin to warrant an opinion of probable causes of failure, but may be due to any of the following: (1) base floated to a smooth surface, precluding possibility of any bond with bituminous material; (2) bituminous material lacks necessary characteristics to enable it to adhere to concrete; (3) base too hard and surface too thin to resist impact of horse-drawn traffic and raveling of surface results.

The danger of failure of bituminous pavements is lessened when care is used in the preparation of specifications and intelligence is shown in the inspection of the work. When failures do occur, they can generally be traced to unsuitable materials, poor foundations, improper machinery, or inferior workmanship. Bituminous concrete mixtures depend to a large extent on their structural stability, which is determined by their density, and by the continuous adhesion of the bitumen-covered particles under traffic, and violation of such well-known principles as grading of the mineral aggregate, consistency of bitumen used, or temperature of mixture and manner of laying, will lead to failure. For bituminous macadam pavements constructed under the penetration method, the quality of the stone, the proper sizing of the stone for each successive course, and the consistency of the bitumen used often determine the success of the pavement. These features were discussed by the speaker in the 1912 Road Meeting of the Society.* It is not considered advisable to enter into a description of the causes of failure of sheet-asphalt, asphalt block, and similar pave-

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 615.

Mr. Spencer.

ments at this time, as this subject is fully covered in the discussions of Societies giving particular attention to these classes of pavements.

Bituminous surfaces originally treated with cold bituminous material may be renewed at slight cost. The extent of sweeping such surfaces is governed by the area and character of the existing cover on the road. If all loose dirt is swept from the original surface, and the bituminous material is covered with clean sand or screenings of a character not readily ground up by the traffic, then a re-treatment without the necessity of sweeping may safely be attempted. Roads properly cleaned, oiled, and sanded should leave a coating of bituminous material to which the fresh material can readily attach itself. The quantity need not exceed ⅜ gal. per sq. yd., and if sand is required, not more than 10 lb. per sq. yd. should be used, or a mushy, soft condition will follow. The bituminous material should be of such consistency that it can be readily applied by a pressure distributor, and should contain a minimum of material likely to lubricate the stone base. Where more than one application per season is required, the road should be treated in the fall, in order to assist as much as possible in maintaining the surface through the winter.

The most successful hot oil or blanket treatment, in the speaker's opinion, is that which has been done in Massachusetts; and, on many miles of roads which he has been over in that State the problem of maintenance has been solved successfully. Mr. Farrington has carefully gone into the methods of maintaining this type of surfacing, and detailed descriptions, extending over a period of 4 years and given in previous publications of this Society by Messrs. A. W. Dean, F. C. Pillsbury, and J. A. Johnston, should serve as a model in those communities which really desire economical maintenance on roads subjected to heavy automobile traffic. For roads treated by the hot blanket method and showing the holes due to settlement of foundation, heaving by frost action, or wearing away, caused by lack of adherence to the base, the earlier such defects are repaired the longer will be the life of the treatment. After 2 years' service of a treatment of this nature, during which time not more than one-third of the surface should be in need of repairs, it becomes a question of the advisability of re-treating the entire surface or of tearing it up and resurfacing with a more permanent pavement. For the repair of minor holes, a well organized gang can readily keep a stretch of road in perfect condition at a moderate outlay for plant and materials. Such a repair outfit should have a portable kettle, for heating the bituminous material, and a compartment for keeping dry the grits or screenings. The hole to be repaired should be swept clean and painted with hot bituminous material of the same consistency as that used in the original treatment, and then filled with grits or screenings to the level of the surrounding surface. This should then be grouted with bituminous material and covered with a

Mr. Spencer. light layer of stone chips or gravel. Where practicable, the patches should be rolled, and should be carefully watched to see that the traffic does not tear them out before they properly set. Another method of repairing such bituminous surfaces successfully is by mixing the aggregate and bituminous material away from the site of the road and depositing them in the hole to be repaired. This should be tamped, sanded, and thrown open to traffic. The speaker believes this method leads to more uniform results.

Bituminous Pavements.—The maintenance of the older types of bituminous pavements has been sufficiently studied to need no elaboration here. This refers to bituminous concrete, sheet-asphalt, asphalt block, and rock asphalt pavements. The more recent bituminous macadam pavements constructed under the penetration method in many cases show the need of immediate repairs. A study of the causes responsible for such failure should be made with a view of obviating these defects in the original construction. The speaker, however, does not at all agree that the penetration method is the total failure that some authorities would have us believe, but is convinced, on the contrary, that for certain kinds of traffic this type of pavement is good enough. This conviction is based on the features of low first cost, length of service, ease of repair, and moderate charge for plant rental with which to apply the bituminous materials properly. Penetration pavements have now been down for a period extending through five summers and winters, have had no repairs spent on them, and are still in perfect condition. Where failures have occurred they can generally be traced to some defect in the original construction. The main causes of such failures are: (1) improper foundation; (2) improper sizes of mineral aggregate; (3) sealing of top course before applying bituminous material; (4) bituminous material of wrong consistency; and (5) uneven application of the materials. The various ways that failures occur in work of this nature are shown on the surface by: (*a*) wavy condition, showing either an unequal distribution of bituminous material or lack of bond to the underlying stone; (*b*) depressions in the surface, due to settlement of the foundation; (*c*) raveling, due to the bituminous material not adhering to the stone; (*d*) surplus of bituminous material on the surface, due to the difficulty of penetrating the stone, combined with improper consistency. For patching holes in a penetration road it is best to clean it thoroughly, scarify the surface, and add sufficient stone to bring the top course to the level of the surrounding road. The heated bituminous material is then applied, and stone of the next smaller size is rammed into the interstices. Generally, only one application is required, and care should be taken that the patch is at the same level as the surface of the road. Where there is an extreme wavy condition, it would probably

be found more advisable to scarify the entire surface, adding sufficient stone, if necessary, and treating it with a light application of bituminous material followed with screenings and thorough rolling. Where raveling has taken place, a blanket treatment of bituminous material, applied under pressure, will often seal the road, without the necessity of scarifying. Mr. Spencer.

Bituminous concrete pavements laid cold (including bituminous mortar) are ordinarily maintained with material prepared at some central point and shipped to its destination. This may be used to repair holes or to spread over the surface to form a new roadway.

Equipment.—For the application of bituminous material, by far the most successful method is by the pressure distributor. This may be horse-drawn, operating a small pump by a sprocket chain, or the pump may be driven by a gas engine in the rear of the wagon; but the most economical is the auto-driven sprayer. The speed can be regulated to spread any quantity required in a uniform spray, and under uniform pressure. For the blanket treatment, there has recently been placed on the market a machine designed to apply heated materials under pressure. This is considered the most advanced and improved method of applying heated material, and its use should be encouraged wherever possible. The outfit consists of a double-shell, asbestos-lined tank, holding from 750 to 1 000 gal., and containing 1½-in. heater pipes. A boiler, equipped to burn either coal or fuel oil, is carried on the rear. This furnishes steam to heat the bituminous material in the tank and to operate an air compressor. The bituminous material is forced through the outlet nozzles under the pressure desired, the quantity being gauged by the speed of the truck and the size of the outlets. This machine can apply the heavier material used in the construction and maintenance of bituminous pavements under the penetration method, and can readily apply the seal coat on bituminous concrete pavements. Where long stretches of road are in need of repairs, the auto-truck sprayer will be found the most economical, and where communities do not care to invest in the costly machines of this nature their services can be contracted for, on a gallon or yardage basis, from the companies operating them.

Indirectly related to the question of maintenance is the uniformity of the bituminous material to be used. There have been discussions at various times relating to the physical and chemical requirements of bituminous materials, and specifications, designed to limit and control their manufacture, have been drawn. Speaking only in reference to petroleum asphalts, with which the speaker is more familiar, a few of the features governing the manufacture of the products under given specifications will be cited as illustrating some of the difficulties attending their preparation. In the first place, for a given grade of crude oil, a uniform product should result from the manu-

Mr. Spencer. facturers' plant. On the other hand, elaborate tests may be interesting, as a matter of record and to classify the asphalt used in any given piece of construction, but when the manufacturer is asked to meet a variety of tests, necessitating a change in his process, a hardship is imposed on him, with no betterment of the resulting product.

The manufacturer who uses stills in the preparation of his material is governed largely by three fundamental conditions: (*a*) the character of the base material he is refining; (*b*) the temperature of distillation; and (*c*) the time of distillation. For a given crude supply, which may be considered satisfactory for the preparation of asphalt, it will be seen that the resulting product is dependent largely on the temperature and the time of distillation. In many cases any change in one to meet a given test will affect seriously more important and necessary tests. As an illustration may be mentioned the fixed carbon requirement in certain specifications. It is agreed, on excellent authority, that the fixed carbon in an asphalt is the product found by dividing the fixed carbon in the crude material by the yield. Therefore a blanket clause in specifications limiting the percentage of fixed carbon, without taking into consideration the crude material which is being run, imposes a condition which the manufacturer is powerless to meet. He cannot control the minimum of fixed carbon, which is governed entirely by the yield and percentage in the crude, and a certain leeway should be given him for the maximum, to allow for errors in the test, imperfections in the process, and causes over which he has no control. Temperatures may control this maximum; but, in making a product for the highest class of paving work, there is no incentive or desire on the part of the manufacturer to injure the product in any way possible, and he would have no object in running at a temperature other than that which will produce the best material.

Engineers are prone to adopt technical specifications for materials used by other communities without investigating their merits as applied to their own work. Although this may be taken as a compliment to the originators of such specifications, in many cases it is done through a lack of knowledge of the materials to be used, whereas the unbiased judgment of those qualified to speak, with an eye to the considerations of cost and durability, will aid many engineers in the intelligent preparation of specifications.

The opinions of engineers assembled to discuss almost any form of construction and tests on most grades of materials will represent the most up-to-date and best thought, and engineers cannot go far astray in adopting such specifications, provided they have been tried and in actual use. Therefore national societies have a duty to perform in perfecting specifications for paving materials which are to be used generally throughout the country, and care should be taken that such specifications contain only those items which will insure a satis-

factory product for the work to be done and not impose conditions impossible to fulfill with the materials available. Mr. Spencer.

T. HUGH BOORMAN, ESQ.—In the discussion 2 years ago the speaker stated that Nelson P. Lewis, M. Am. Soc. C. E., Chief Engineer of the Board of Estimate and Apportionment of the City of New York, had considered that rock-asphalt pavements were probably the best that had been laid. George W. Tillson, M. Am. Soc. C. E., objected to it because he considered the maintenance charges on it excessive. The speaker, therefore, begs to state that rock-asphalt comprimé has been laid in the United States in New York, Brooklyn, Long Island City, and Rochester, in New York State; Elizabeth and Perth Amboy, in New Jersey; Boston, Mass.; New Haven, Conn.; Philadelphia, Pa.; and New Orleans, La. Mr. Boorman.

The first comprimé work was laid in Union Square, New York, in 1872. In August, 1897, 112th Street, from Fifth to Lenox Avenues was laid, and the city records show that in 1912 the cost of repairs on this street was only 6 cents per sq. yd.; and 101st Street, from Lexington Avenue to Park Avenue, paved in July, 1896, with Mons and Sicilian rock, bears no cost for repairs from 1910 to 1913. Such durability seems to be unparalleled in the history of street construction. On Dyckman Street, from Kingsbridge Road to the tracks of the New York Central and Hudson River Railroad, 8 000 sq. yd. of Seyssel and Sicilian rock asphalt were laid in the late fall of 1897, and the cost of maintenance for the 4 years, 1910 to 1913, inclusive, was 3 cents per sq. yd., or less than 1 cent per sq. yd. per year. In 1901, between Park and Lexington Avenues, 35th and 36th Streets were paved with Seyssel and Sicilian rock asphalt; on 36th Street no repairs were necessary for the 4 years, 1910-13. In the same year 13th Street, from Second to Third Avenues, was paved with Sicilian and Mons rock on an old stone foundation, and is charged with 14 cents per sq. yd. for 4 years' maintenance. In 1897, 106th Street, from Broadway to Riverside Drive was paved with Sicilian and Mons rock, and shows cost for repairs of 17 cents per sq. yd. for 4 years, a trifle more than 4 cents per year.

To come to more recent times, it was decided in 1912 by the Department of Highways, under the advice of E. P. Goodrich, M. Am. Soc. C. E., Consulting Engineer to the President of the Borough of Manhattan, to lay a series of test pavements on Second Avenue, from Houston to 23d Streets. Rock asphalt comprimé, rock-asphalt blocks, and Sicilian rock comprimé were laid between 19th and 21st Streets, and on inspection, in January, 1914, these two blocks were found to be in good condition, the only imperfection being in the work adjoining the car tracks. This is not to be wondered at, as all rock asphalt experts, such as Malo, Delano, Walsh of Amsterdam, Bassett of London, and almost all engineers in Europe have decided that granite or other

Mr. Boorman.

blocks should be laid longitudinally beside car tracks. The speaker recommends scoria blocks. The most noticeable of all the experimental pavements on Second Avenue is that of rock asphalt compressed blocks, between 9th and 11th Streets. These blocks were manufactured, at the Staten Island Works of the Sicilian Asphalt Paving Company, in an ordinary German brick machine, and did not receive the perfect compression obtained by the Val de Travers Asphalte Paving Company in their works at Marseilles, Seyssel, Cairo, Egypt, and other places. However, the rock being pure crude rock asphalt powder before being pressed, simply spread under heavy traffic, and became virtually a monolithic sheet of rock asphalt, and it affords to-day as perfect a specimen of rock asphalt pavement as can be seen in any country.

Mr. Delano, on the occasion of his visit to New York in November, 1913, stated that the City of Paris had recently signed contracts for replacing wood blocks, stone sets, and macadam with 700 000 sq. m. of rock asphalt pavement, to be laid during the next 5 years. Attention is also called to the fact that, some years ago, Thomas Street, from Broadway to Church Street, and Trimble Place, from Thomas to Duane Streets, both private streets owned by fifty associates, were paved with Neuchatel Rock Asphalte Pavés and coated with an asphalt rubber surface coat, only ⅜ in. thick, which has stood the heavy traffic of those busy thoroughfares for several years.

The speaker can prove that rock asphalt is the most valuable pavement in the United States. This is stated especially because so many engineers doubt that any European rock asphalt, such as laid in London, Paris, and Berlin, has ever been laid in the United States. The speaker is sorry that, owing to the short time allotted to him, he will not have the opportunity to say a little about its history in America.

The reason this subject of the practicability of the use of European Rock Asphalte is so much more important than when the discussion before the Society was held in 1912, is that the tariff on asphalt has been removed, and Rock Asphalte powder can now be purchased at a cost of $3 per ton less than in 1913, making a difference of 25 cents per sq. yd. in the cost of the surface. Municipal plants can be supplied with the material, all ready for heating and compressing on the street, and no additional outlay for machinery is necessary. Both the Borough of Manhattan, which is now completing its new municipal asphalt plant, and the City of Philadelphia are considering the advisability of doing their own repairs to rock asphalt streets.

Mr. Sharples.

PHILIP P. SHARPLES, ESQ.*—One or two points brought up in the discussion were not entered into very fully. For a number of years the speaker has been very much interested in studying the effect of oil on tar-bound macadam pavements. In considering the subject,

*Chf. Chemist, Barrett Mfg. Co., Boston, Mass.

country road practice and city or town practice must be sharply differentiated. Mr. Sharples.

In building country roads, such as those described by Mr. Farrington, it has been the practice in many cases to construct a tar-bound macadam road, and coat it with one of the heavy asphaltic oils or light asphalts put on hot. This practice has given excellent results in many cases, and has been established as a standard specification in some States, notably in Connecticut. Some very good work has been done in this way.

In carrying out work under a specification of this kind, it is very important that the grade of oil or oil asphalt be so heavy that it will not penetrate the tar-bound macadam, but will stay on top as a blanket layer. An oil light enough to penetrate the tar-bound road is sure to give bad results. The speaker knows of a number of roads of this type, which were well built and then treated with a light oil, while still presenting a porous top. They disintegrated completely within a year when subjected to heavy horse-drawn traffic, although neighboring sections sealed with tar products remained in good condition. If, however, a proper grade of asphaltic oil is used, this disintegration will not occur.

Under city or town traffic conditions, the tendency of an oil-asphalt top over a tar-bound macadam is to mush up and become soft under horse-drawn traffic in wet weather, due to the emulsifying effect of the water and organic material which collects on the asphalt top. When these conditions are to be provided for, it is important, so far as present experience indicates, to use only tar seal coats on top of the tar-bound macadam. The refined tars used for seal coats are preferably somewhat softer than the tar used in the tar-bound macadam, but the subsequent treatment and care of the road modifies somewhat the character of the seal coat required.

For maintenance within city and town limits, where the horse-drawn traffic is considerable, it has been found that a specially prepared tar, which can be applied cold, gives much better results than the heavier tar materials put on hot.

There are records of streets, in the vicinity of Boston, which carry more than 4 000 vehicles per day, which have been kept in good shape since 1908 with an annual maintenance cost of about 3 or 4 cents per sq. yd., with prepared tar applied cold.

Consider the light dust-laying oils and their effect on tar-bound macadam. It is a problem which affects city and town conditions. In country practice, the conditions do not arise where an oil of this type is necessary. On city and town streets, a troublesome dust often collects, which the ordinary seal coat of tar or heavy asphalt does not keep down. In applying a light oil over a tar-bound macadam to keep down this superficial dust, it is essential to comply with certain

Mr. Sharples. conditions if the tar-bound macadam is not to be ruined. It is the utmost folly to put a light oil on a tar-bound macadam; if there is the least chance for the oil to penetrate the macadam, the results are sure to be disastrous. The tar-bound macadam will be disintegrated. A light oil has two effects on the tar-bound macadam. It has a chemical effect on the tar which causes its disintegration and makes it lose its binding force. It also has a physical effect on the macadam structure. The oil is a lubricant; and if there is any stone loose enough to move in the tar macadam, the light oil accelerates the movement, and a hole is sure to develop. This dual action of the light oil can be observed in many large cities at the present time, and many streets may be seen which have been built with tar-bound macadam, the life of which has been gradually shortened by the use of light oil.

The effects of the light oil can be minimized if the tar-bound macadam is thoroughly sealed, either through an excess of bituminous material in the first place, or by subsequent applications of refined tar. Certain precautions must be taken also in applying the oil. An excess of oil should never be used, and it should be applied to the surface only in sufficient quantity to take up the loose dust which is found there. A very small fraction of a gallon is sufficient. If more than this is applied, the oil gradually sinks into the tar-bound macadam and in the end will rot it.

Wherever a light oil treatment is used on a city or town street for dust-laying purposes, it is quite essential to revivify the surface of a tar-bound macadam from time to time by the application of a tar seal coat. For this purpose nothing seems to be better than a prepared tar of such consistency that it can be applied cold. Only a very small quantity is required, usually not more than ¼ gal. per sq. yd. per annum. If these applications are put on intelligently, and if patching is intelligently attended to, a tar-bound macadam will give good service, even in the face of light oil applications.

Mr. Washington. WILLIAM DE H. WASHINGTON, ASSOC. M. AM. SOC. C. E.—Three members of this Society were engaged by the Highway Department of the State of New York to devise methods of maintaining highways and reducing the cost of maintenance. They concluded that the railroads had set a very excellent example as to the organization for upkeep, by which they put this work in the hands of a body of men under a competent foreman, instead of distributing their men individually over a section. These men work as a unit. Each group of men is provided with all necessary materials, which materials are usually carried on hand-cars.

The speaker has devised a machine to accomplish exactly the same object in connection with the maintenance of roads, which meets the requirements mentioned in Mr. Farrington's paper. This movable

repair plant consists of a motor truck equipped with all tools and machines necessary for the maintenance of highways, and supplied with storage capacity for broken stone, sand, and tar. The machines include a pressure spraying device, sand and stone chip distributor, a roller (beneath the body of the truck), a stone and sand dryer, a mixing machine, a road sweeper, a revolving power scarifier, extension arms for pulling a plow, scraper, split-log drag, or a snow-plow, a power loader and derrick, and all requisite small tools. The truck has storage capacity for about 4 tons of broken stone and from 12 to 14 bbl. of tar. When used for the repair of macadam roads, a large removable water tank is carried on the truck. Mr. Washington.

The outfit has been designed with the intention of utilizing it in the maintenance of all types of roads and pavements constructed by the State Highway Department of New York.

A. F. Masury, Esq. (by letter).—There is no equipment better adapted for the maintenance of bituminous surfaces and bituminous pavements, or for making roads, than motor trucks; and probably none is more generally in use for this purpose. Mr. Masury.

This discussion will give facts, based on the actual operation and maintenance of trucks, which will enable the engineer and contractor to operate them to the best advantage in road work.

Factors of Motor Truck Operation.—Glaring generalities have been characteristic of most discussions on motor truck operation. The motor truck salesman, with his optimistic figures and his aggressiveness to make a sale, has caused the engineer, contractor, and merchant to use his figures in planning truck operation. This, done without much thought on the part of the owner, really gives him the salesman's figures of past performances on which to base his intended operation.

The commercial motor vehicles are machines, made in a variety of types and sizes, designed for specific purposes, and must be built, their field of operation laid out, their operation controlled, and the up-keep looked after, by engineers.

The purpose of this discussion is to present data on different sizes and types of motor trucks, to show actual figures as to their performance, and state the factors which must be used in operating them.

Arrangement of Space for Most Economical Storage.—In storing motor trucks, it has been worked out in practice that they can be handled best in a building of usual size by arranging them in two rows with an open aisle between; that is, the trucks are backed into place, with their fronts facing the aisle. This permits them to be taken out without any delay, and also allows them to be run into the aisle, where washing and drainage facilities should be provided.

Mr. Masury.

In most buildings, posts are necessary, and should be toward the front of the trucks, but far enough back to allow the trucks to turn out into the aisle. The depth from the wall to the post should be at least 16 ft. 6 in.; thus, the front wheels of trucks of most sizes will project beyond the posts and allow an easy and quick turning into the aisle. The aisle should be about 27 ft. wide in order to provide a practical turning radius for getting out or backing in. The posts should be about 22 ft. apart, so as to allow the storage of three large trucks between them. Where a large number are stored the proportioning of small to large trucks allows this arrangement to be worked out with the economical use of all space.

The size of the doors should be looked into very carefully, as trucks, especially of the van type, are now very large. In order to take those of the largest type, such as vans and those used for crackers and paper boxes, the doors should have a clearance of at least 12 ft. 3 in., and should be double, opening outward. The clearance between the butts of the doors should be at least 10 ft. 9 in. This allows sufficient space for the fastenings of the doors, without interfering with the trucks as they move in and out. There should be lights on the butts of the doors on each side, quite high up.

The elevators must also be considered carefully. They must be at least 24 ft. long and 11 ft. wide, in order to take the modern motor truck, and should have a carrying capacity of at least 7 tons, in order to accommodate the trucks when light. A heavier elevator, to take care of loaded trucks, is impractical, from a commercial standpoint; loaded trucks are not usually garaged in such buildings, but might be accommodated on the ground floor. The elevator should have four guiding rails, and a speed of about 60 ft. per min.

Service.—There must be adequate facilities for filling the tanks with gasoline and oil, and for the storage of the large quantities of lubricants used in the operation of such trucks.

To comply with fire prevention regulations, the gasoline must be stored underground, and must be piped to at least two outlets on each garage floor. There are two practical ways of raising it to each floor: by hydraulic power and by pressure. Either of these systems can also be used for cylinder oil and kerosene. There should be at least one outlet for kerosene and one for oil on each floor. Meters should be attached to all outlets, and provision should be made for locking them. At least one portable tank should be provided for each floor, as trucks must be filled during the hours when the garage is most crowded, and when, at best, only two can be filled from each outlet.

Oil and kerosene may be carried to the trucks from the outlets in measures, as comparatively small quantities are used.

Transmission oil, cup grease, wood alcohol (for use in the radiators in cold weather), and other material and accessories may be stored in a room on the floor with the trucks. Mr. Masury.

There should be an efficient system for keeping account of the exact quantities of supplies consumed by each machine.

A dirty truck (and trucks collect dirt with astonishing rapidity) is much harder to keep in good order than a clean one, as dirt not only gets between moving parts and into bearings and causes wear, but it prevents the discovery of minor troubles until they have become major ones, and makes repairs a difficult and distasteful job if they are discovered. Therefore, trucks should be washed at night. For this purpose a regular force should be employed, its size depending on the number of cars to be washed. Four washers and two polishers can take care of fifty cars.

Drivers are not always to be depended on to make repairs; their hours are long, and they have little time for them during the day's run. For this reason, if six or more trucks are being operated, it will be well to have a night repair man on duty, with one or two helpers. As the trucks come in, each driver should report any trouble with his machine. The repair man can also examine the trucks to discover loose bolts, and can grease them up.

Truck Sizes.—Trucks may be divided into three classes, according to the arrangement of the motor and the driver's seat:

Type 1.—This is the usual touring car type; the motor is over the front axle, the driver and control are behind the motor.

Type 2.—In this type the motor is over the axle; the driver and control are directly above the motor.

Type 3.—The motor is over the front axle; the driver and control are on one side of the motor, and the helper is on the other side.

Types 2 and 3 conserve the length of the chassis, as less space is taken for the driver and control, thus allowing a longer loading platform in proportion to the wheel base than Type 1.

The most important average dimensions for each size of truck are given in Table 2; and the diameters of turning circles are given in Table 3.

Factors in Cost of Operation of Motor Trucks.—Table 5 shows the cost of operation of trucks of standard capacities running at different mileages per day, and the factors in detail of each item of up-keep. These figures are taken from actual costs of operating trucks, and are nominal in every way. Interest is reckoned at 5% on half the cost of the truck; insurance at 2% on 80% of the cost; and amortization at 10 per cent.

Mr. Masury.

TABLE 2.—MOST IMPORTANT AVERAGE DIMENSIONS OF TRUCKS.

Type.	Capacity.	Wheel base.	Extreme length.	Length, back of cab.	Extreme width, in inches.	Height of Loading Platform, in Inches. Empty.	Height of Loading Platform, in Inches. Loaded.	Clearance, in inches.
Type 1	1-ton.	12 ft.	16 ft.	9 ft. 2 in.	68	36	32	9½
	2-ton.	13 ft.	17 ft.	9 ft. 2 in.	68	38	34	9½
	3-ton.	13 ft. 6 in.	20 ft. 6 in.	11 ft. 0 in. to 11 ft. 6 in.	87	42	38	10
	4-ton.	14 ft. 6 in.	21 ft. 6 in.	12 ft. 0 in. to 12 ft. 6 in.	87	42	38	10
	5-ton.	15 ft. 6 in.	22 ft. 6 in.	13 ft. 0 in. to 13 ft. 6 in.	87	44	38	10
	7½-ton.	16 ft. 6 in.	23 ft. 6 in.	14 ft. 0 in. to 14 ft. 6 in.	87	44	38	10
Type 2	3-ton.	11 ft. 6 in.	18 ft. 3 in.	12 ft. 6 in.	87	42	38	10
	4-ton.	12 ft. 6 in.	30 ft. 3 in.	14 ft. 6 in.	87	42	38	10
	5-ton.	13 ft. 6 in.	22 ft. 3 in.	16 ft. 6 in.	87	44	38	10
	7½-ton.	14 ft.	23 ft. 3 in.	17 ft. 6 in.	87	44	38	10
Type 3	5-ton.	11 ft. 10½ in.	18 ft. 1 in.	14 ft.	93½	46	40	7½
	7-ton.	13 ft. 4½ in.	20 ft. 1 in.	16 ft.	93½	46	40	7½
	10-ton	14 ft. 2½ in.	22 ft. 1 in.	18 ft.	93½	46	40	7½
Power-dump truck, Type 1.	5-ton.	11 ft. 9 in.	16 ft. 5 in.	8 ft. 3 in.	87	44	39	10½
	7½-ton.	12 ft. 10 in.	17 ft. 5 in.	9 ft. 3 in.	87	44	39	10½

TABLE 3.—TURNING CIRCLES.

Type.	Capacity.	Wheel base.	Diameter of turning circle, in feet.	Type.	Capacity.	Wheel base.	Diameter of turning circle, in feet.
Type 1...	1-ton....	10 ft. 6 in.	47	Type 1...	5-ton....	12 ft. 6 in.	57
		11 ft. 6 in.	53			13 ft. 6 in.	62
		12 ft. 6 in.	55			14 ft. 6 in.	68
Type 1...	1½-ton..	10 ft. 6 in.	47	Type 1...	7½-ton..	12 ft. 6 in.	57
		11 ft. 6 in.	53			13 ft. 6 in.	62
		12 ft. 6 in.	55			14 ft. 6 in.	68
		13 ft. 6 in.	60			15 ft. 6 in.	71
Type 1...	2-ton....	10 ft. 6 in.	47	Type 2...	3-ton....	11 ft.	47
		11 ft. 6 in.	53			11 ft. 6 in.	49
		12 ft. 6 in.	55			12 ft.	51
		13 ft. 6 in.	60	Type 2...	5-ton....	11 ft. 6 in.	53
Type 1...	3-ton....	12 ft. 6 in.	55	Tractor..	3-ton......	10 ft. 4 in.	40
		13 ft. 6 in.	59	Tractor..	5-ton......	10 ft. 8 in.	41
		14 ft. 6 in.	61				
Type 1...	4-ton....	12 ft. 6 in.	55				
		13 ft. 6 in.	59	The trailer will usually cut under.			
		14 ft. 6 in.	61				

In Table 5 the cost of the body is left out intentionally, as it usually represents the amount of discount given by the manufacturer, and it needs but few repairs.

Tire Data.—The data on tire equipment in Table 4 show the average of the requirements and guaranties of the different tire man-

Mr. Masury.

ufacturers, and also the average cost of tires to the consumer. Column 2, the axle load, gives the average weight on the front and rear axle of trucks of each size. Column 3, the size of tire, shows the sizes usually recommended by tire manufacturers for the respective loads carried on each axle. Column 4, $\frac{L}{W \times C}$, means the load on the axle, in pounds, divided by the width of the tire (the width being taken at the place where the tire meets the ground); C is the circumference of the wheel. The figures in this column are in pounds.

TABLE 4.—TIRE EQUIPMENT FOR TRUCKS.

(1) Truck Capacity.	(2) Axle load.	(3) Size of tires.	(4) $\frac{L}{W \times C}$	(5) Speed, in miles per hour.	(6) Mileage.	(7) Consumer's price.
1-ton	Front 2 600	36-4 single	18	16.5	7 000	$ 54.15
	Rear 5 200	36-5 single	28.8			70.45
	Rear 5 200	36-2½ dual	28.8			51.30
1½-ton	Front 2 800	36-4 single	19.4	15	7 000	54.15
	Rear 6 300	36-3 dual	29.2			68.80
2-ton	Front 3 060	36-4 single	20.8	14	7 000	54.15
	Rear 7 200	36-3½ dual	28.6			83.67
3-ton, Type 1	Front 4 300	36-5 single	23.9	12½	7 000	70.45
	Rear 10 800	36-4 dual	37.5			108.35
	Rear 10 800	42-4 dual	42.1			126.17
3-ton, Type 2	Front 5 950	36-5 single	33.0	12½	7 000	70.45
	Rear 9 600	36-4 dual	33.3			108.35
	Rear 9 600	42-4 dual	28.5			126.17
4-ton	Front 4 950	36-6 single	22.9	11	7 000	87.97
	Rear 12 950	36-5 dual	36.0			138.90
	Rear 12 950	42-5 dual	30.8			172.40
5-ton, Type 1	Front 5 300	36-5 single	29.4	10	7 000	70.45
	Rear 15 200	36-5 dual	42.2			138.90
	Rear 15 200	42-5 dual	36.2			178.40
5-ton, Type 2	Front 7 425	36-6 single	34.4	10	7 000	87.97
	Rear 15 425	36-5 dual	42.8			138.90
	Rear 15 425	42-5 dual	36.7			172.40
7½-ton	Front 6 175	36-6 single	28.6	7½	7 000	87.47
	Rear 20 325	36-6 dual	49.3			175.97
	Rear 20 325	42-6 dual	42.2			205.85

This factor takes into account the size of the wheel, and therefore reconciles the fact that a larger wheel can have a tire of a smaller size, and is a better factor to use in comparing tire sizes than the weight, in pounds per square inch, of tire in contact with road.

Column 5 shows the normal speed of trucks of each capacity. Column 6 shows the guaranteed mileage by the tire manufacturers. Column 7 shows the average price at which tires can be bought by the consumer.

Mr. Masury.

Column letter in Table 5.		Formulas.
A	Capacity.	
B	Average cost of chassis, based on list prices of the number of manufacturers indicated by the figures in parentheses.	
D	Miles per hour.	
F	Driver's salary, per year.	
G	Cost of garaging, per month.	
H	Miles per day.	
I	Maintenance (small repairs, etc.), per year.	
J	Gasoline, per year.	
K	Oil and grease, per year.	
L	Tires, per year.	
M	Interest, per life.	$M = R\dfrac{B}{2} \times 5$ per cent.
N	Insurance, per life.	$N = R \times (80\%$ of $B) \times 2$ per cent.
O	Depreciation, per life.	$O = \dfrac{B}{10\% \text{ of } R}$.
P	Cost, per year.	$P = F + 12G + I + J + K + L + \dfrac{M}{R} + \dfrac{N}{R} + \left(O = \dfrac{B}{10}\right)$.
Q	Miles of life at each mileage per day truck is assumed to run.	
R	Years of life.	$R = \dfrac{Q}{W}$.
S	Cost, per life.	$S = P \times R$.
T	Cost, per day.	$T = \dfrac{P}{300}$.
U	Cost of tires, per mile.	$U = \dfrac{L}{W}$.
V	Cost of gasoline, per mile.	$V = \dfrac{J}{W}$.
W	Miles per year.	$W = H \times 300$.

Mr. Masury.

TABLE 5.—FACTORS IN THE COST OF OPERATION OF MOTOR TRUCKS.

A Capacity.	B Average cost of chassis.	D Miles per hour.	F Driver's salary, per year.	G Garage, per month.	H Miles per day.	I Maintenance, per year.	J Gasoline, per year.	K Oil and grease, per year.	L Tires, per year.	M Interest, per life.	N Insurance, per life.	O Depreciation during life.	P Cost, per year.	Q Miles of life at each mileage per year.	R Life of truck, in years.	S Cost, per life.	T Cost, per day.	U Tire cost per mile.	V Gasoline, cost per mile.	W Miles of use, per year.
			$780		40	$120	$300	$120	$360	$408.75	$260.70	$1 626	$2 197.78	100 000	8.3	$18 241.74	$7.33	$0.03	$0.025	12 000
1-ton......	$1 963	19	880	20	60	240	450	190	540	231.75	147.00	833	3 106.78	85 000	4.7	14 601.86	10.36	0.03	0.025	18 000
	(12)		900		90	360	675	270	810	117.76	75.00	426	3 531.78	65 000	2.4	8 476.27	11.77	0.03	0.025	27 000
			780		35	135	276	102	341	533.97	331.00	2 140	2 199.73	100 000	9.6	21 117.41	7.33	0.0325	0.0262	10 500
1½-ton....	2 225	18	880	21	50	270	395	150	488	317.06	202.00	1 270	2 698.73	85 000	5.7	15 922.51	9.00	0.0325	0.0263	15 000
	(12)		900		90	405	710	270	878	133.50	85.00	535	3 728.73	65 000	2.4	8 948.95	12.43	0.0325	0.0262	27 000
			780		35	150	288	75	393	663.33	461.00	2 660	2 361.44	95 000	9.1	21 489.10	7.87	0.0375	0.0274	10 500
2-ton......	2 918	17	830	22	50	300	415	150	563	408.52	288.00	1 818	3 067.44	85 000	5.6	17 177.66	10.22	0.0375	0.0276	15 000
	(8)		900		80	450	660	225	900	246.96	136.00	789	4 026.44	65 000	2.7	10 877.39	13.42	0.0375	0.0275	24 000
			780		30	180	292	75	404	893.75	572.00	3 575	2 233.39	90 000	10.0	22 333.90	7.44	0.045	0.0324	9 000
3-ton......	575	15	830	23	50	360	487	150	678	473.68	308.00	1 989	3 612.39	80 000	5.3	19 145.67	12.04	0.045	0.0324	15 000
	(6)		900		80	540	781	225	1 081	241.31	154.00	1 018	4 329.39	65 000	2.7	11 716.35	14.43	0.045	0.0324	24 000
			780		30	240	360	75	450	1 016.50	650.00	4 066	2 761.31	90 000	10.0	27 613.10	9.20	0.05	0.04	9 000
4-ton......	4 066	14	830	24	50	480	600	150	750	538.94	344.00	2 140	3 666.31	80 000	5.3	19 431.44	12.22	0.05	0.04	15 000
	(3)		900		70	720	840	225	1 025	364.95	195.00	1 210	4 566.31	65 000	3.0	13 698.93	15.22	0.05	0.04	21 000
			800		30	240	406	90	540	1 125.00	720.00	4 500	2 920.50	90 000	10.0	29 205.00	9.74	0.06	0.045	9 000
5-ton......	4 500	12	900	25	50	480	678	180	900	596.25	311.00	2 380	4 072.50	80 000	5.3	21 184.25	13.58	0.06	0.045	15 000
	(10)		1 000		70	720	948	262	1 260	337.50	216.00	1 350	5 124.50	65 000	3.0	15 373.50	17.08	0.06	0.045	21 000
7½-ton,...	5 400	9	800	26	25	255	395	94	750	1 525.50	976.00	5 400	3 367.40	85 000	11.3	38 051.62	11.22	0.10	0.0526	7 500
	(2)		900		40	510	630	150	1 200	837.00	535.00	2 348	4 463.40	75 000	6.2	27 673.08	14.88	0.10	0.0525	12 000
10-ton......	5 500	7	800	28	25	270	468	94	1 200	1 553.75	994.00	5 500	3 943.50	85 000	11.3	44 561.55	13.15	0.16	0.0624	7 500
	(1)		900		40	540	750	150	1 920	852.50	545.00	3 410	5 371.50	75 000	6.2	33 303.30	17.90	0.16	0.0625	12 000

Mr. Farrington.

WILLIAM R. FARRINGTON, M. AM. SOC. C. E. (by letter).—The writer notes that considerable attention has been given by those taking part in the discussion on "Methods and Equipment for Maintaining Bituminous Surfaces and Bituminous Pavements" to the use of motor equipment.

Not much has been done, as yet, with trucks in State highway maintenance work in Massachusetts. Trucks have been used in some cases, but, except for transporting broken stone, gravel, bituminous materials, etc., over long hauls, apparently have not demonstrated their economy under existing conditions. The writer has no doubt, however, that, as this work increases and longer sections come under maintenance, motor trucks with the necessary equipment for patching, etc., will prove economical and will come into general use.

As to the material truck described by Mr. Washington, though the writer has no doubt that this will be useful, there would seem to be some objection to its general adoption at present, as it is probable that many of the culverts and bridges, even on main lines, are not strong enough to carry a truck of this weight, and there is also some question as to the effect on the road surfaces.

Recognizing the injurious effect of very heavy trucks on roads, where the surfaces and foundations, as well as culverts and bridges, are not especially designed for such traffic, the Massachusetts Legislature in 1913 passed a law prohibiting the use on the highways of trucks weighing more than 14 tons, without a permit from the State, city, or town officials having charge of repairs on the highways, and it would not seem to be good policy for the State to use on its work trucks of greater weight than the public generally is allowed to use.

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Paper No. 1304

MEASUREMENT OF THE FLOW OF STREAMS BY APPROVED FORMS OF WEIRS WITH NEW FORMULAS AND DIAGRAMS.*

DETAILS AND SUMMARIES OF THE RESULTS OF EXPERIMENTS BY FRANCIS, BAZIN, FTELEY AND STEARNS, AND AT THE HYDRAULIC LABORATORIES OF CORNELL UNIVERSITY AND THE UNIVERSITY OF UTAH.

BY RICHARD R. LYMAN, ASSOC. M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. E. A. MORITZ, ALLEN HAZEN, CLARENCE T. JOHNSTON, R. B. ROBINSON, L. M. WINSOR, H. W. KING, ROBERT E. HORTON, G. E. P. SMITH, CLARENCE S. JARVIS, GARDNER S. WILLIAMS, AND RICHARD R. LYMAN.

It is probable that in the near future practically every farmer in the West will be required to place a weir at the head of his farm for the purpose of measuring the irrigating water he uses.

Wherever water is used for any purpose, its measurement, in a more or less accurate way, is important; but in the middle and western parts of the United States, and wherever else practically all agricultural interests depend on water for purposes of irrigation, this problem is of the utmost importance. Water has already become so valuable in these sections that laws have been enacted requiring that weirs be placed in every important stream.

If only the matters herein discussed are considered, the title given to this paper is a broader one than it deserves; but the method of

* Presented at the meeting of November 19th, 1913.

handling the weir data is such that, in the writer's opinion, it may wisely be applied to every phase of the weir problem.

In the West the measurement of water should be made in the most accurate manner practicable. From the point of view of the engineer and the investigator, the method of calculating the discharge over a weir should be the simplest that can be found; in other words, it should give the most accurate results with the smallest amount of work.

It is the purpose of this paper to present a more accurate method of measuring water than those generally used, and to give formulas and diagrams for determining the discharge over the weirs recommended. These, the writer believes, are simpler and easier of application than any weir formulas heretofore devised.

Soon after the publication of the handy and thoroughly practical "Diagrams of Mean Velocity of Uniform Motion of Water in Open Channels,"* by I. P. Church, Assoc. Am. Soc. C. E., of Cornell University, the writer conceived the idea of preparing a similar set of diagrams for determining the quantity of water flowing over weirs under various conditions. The results of his efforts are presented as a part of this paper.

The use of the various diagrams will be explained first; afterward, the data on which they are based will be set forth, and then the methods used in getting the results.

I.—Flow of Water Over Weirs.

A.—Advantages in Practice of Using Weirs Without End Contractions.

First.—The water above the weir without end contractions has a greater velocity for the same quantity of flow than above weirs with end contractions, therefore less sediment will be deposited above the former than above the latter. This fact is important, as the accuracy of the measurements obtained depends largely on having the cross-section of the stream above the weir constant, especially if the height of the crest of the weir above the bottom of the channel of approach is comparatively small.

Secondly.—The weir without end contractions will necessarily have to be placed near the down-stream end of a channel made of lumber or some material which will retain a rectangular form. Although this requires a comparatively expensive construction, the weir and the

* Published by John Wiley and Sons, New York.

bulkhead holding it can be removed, thus making it easy to flush and clean the channel. In fact, if a very high degree of accuracy is not required, the bulkhead and metal crest need be put in place only when measurements are actually being taken.

Thirdly.—As the quantity of water flowing over the weir is almost exactly proportional to its length, water may be divided accurately at the same time that it is being measured, by placing one or more vertical dividing blades below the weir and cutting the stream into the desired number of parts having the required proportions. Where end contractions are used, they affect the quantity of discharge per unit of length of weir in such a variable and uncertain way as to make an accurate division of the water as it falls very difficult, if not actually impossible.

Fourthly.—The extensive and accurate information herein given, and used as a basis for the results reached, has been obtained by our greatest hydraulicians. We may have to wait a long time before obtaining a series of experiments on any form of weir which will equal these in thoroughness, extent, or accuracy.

Fifthly.—The simplicity of the correct construction of this weir is a great advantage. The crest is its only sharp edge. It is easier to make the sides plane and vertical than to construct them on a definite slope with sharp edges, as is necessary, for example, in the Cippoletti weir. Almost any farmer, with simple instructions, can construct a weir of this type which will answer well for measuring the water he uses on his land.

Sixthly.—The simplicity of the method of finding accurately the discharge over the weir, by using the diagram herein given, is perhaps not the least of the advantages the weir without end contractions has over others. The method of ascertaining the discharge is so simple that any one who can read the depth of the water on the crest can read the discharge from the diagram; and as the only mathematical operation required is the multiplication of the figure thus obtained by the length of the weir, in feet, the chance of making an error is reduced to a minimum.

It is the writer's opinion that the sharp-crested weir without end contractions can certainly be used to best advantage in all irrigation projects, great and small, of the West, where mountain streams must soon be measured, divided, and re-divided many times before the water

is finally used for producing crops. It will be well, indeed, if some legislation can be enacted soon, or some other action can be taken, which will make this weir a standard for measuring water, just as the House of Representatives of the United States fixed a standard gauge for measuring the thickness of sheet iron and steel.

B.—Application of Results to:

1.—Sharp-Crested Weirs Without End Contractions.

a.—For Heads Within the Limits of the Experiments Performed.— Plate XXI.—The curves on Plate XXI, which show the discharges over sharp-crested weirs without end contractions, are based on experiments by Bazin, Fteley and Stearns, Francis, and in the Hydraulic Laboratories of Cornell University and the University of Utah. As this diagram is constructed from the results of all these "classic" experiments, and as it fits the experiments with a high degree of accuracy, the writer feels that its results can be relied on with greater confidence than can those obtained by using any one of the more or less complicated weir formulas which the experimenters themselves have devised, using only their own experiments as a basis.

Although, by use of the analytic expressions or formulas given later, quantities of discharge can be computed containing a greater number of significant figures than can be read from this diagram, and from others herein presented, these easily give results as accurate as warranted by the experimental data on which they are based. See Plate XXII.

On Plate XXI there are three sets of lines, each marked so as to be easily distinguished from the others. One represents the head on the weir, in feet, another the discharge over the weir, in cubic feet per second per foot of length of weir, and the third the height of the crest of the weir above the bottom of the channel of approach.

Example 1.—What volume of water per second will flow over a sharp-crested weir, 5 ft. long, without end contractions, the height of the crest above the bottom of the channel of approach being 2 ft., and the depth of water on the crest (the height of the surface of the water above the crest) being 1.20 ft.?

The line representing a head of 1.20 ft. on Plate XXI intersects that for a weir of height 2 ft. above the floor of the channel of approach

FIG. 1.—DURING THE HIGH-WATER SEASON.

FIG. 2.—DURING THE LOW-WATER SEASON.

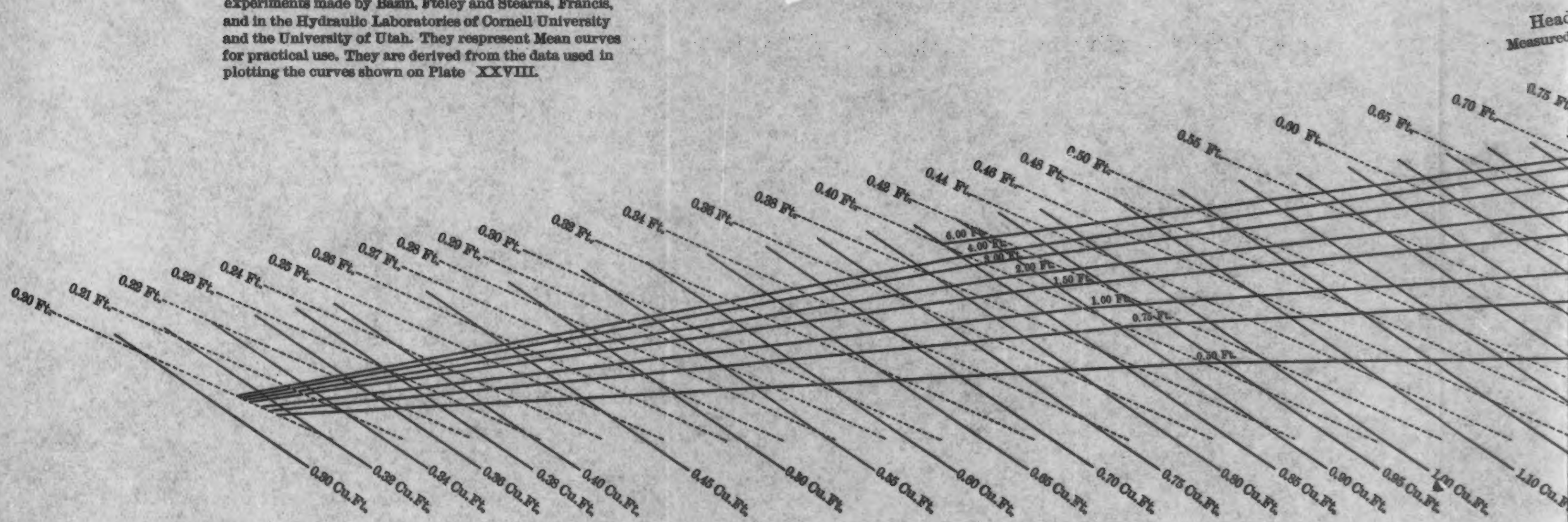
THE HYDRAULIC LABORATORY OF CORNELL UNIVERSITY.

FIG. 1.—DURING THE HIGH-WATER SEASON.

FIG. 2.—DURING THE LOW-WATER SEASON.

GENERAL DIAGRAM OF HEADS AND CORRESPONDING DISCHARGES OVER SHARP-CRESTED WEIRS OF VARIOUS HEIGHTS WITHOUT END CONTRACTIONS.

Note:- These curves are derived from the results of experiments made by Bazin, Fteley and Stearns, Francis, and in the Hydraulic Laboratories of Cornell University and the University of Utah. They respresent Mean curves for practical use. They are derived from the data used in plotting the curves shown on Plate XXVIII.



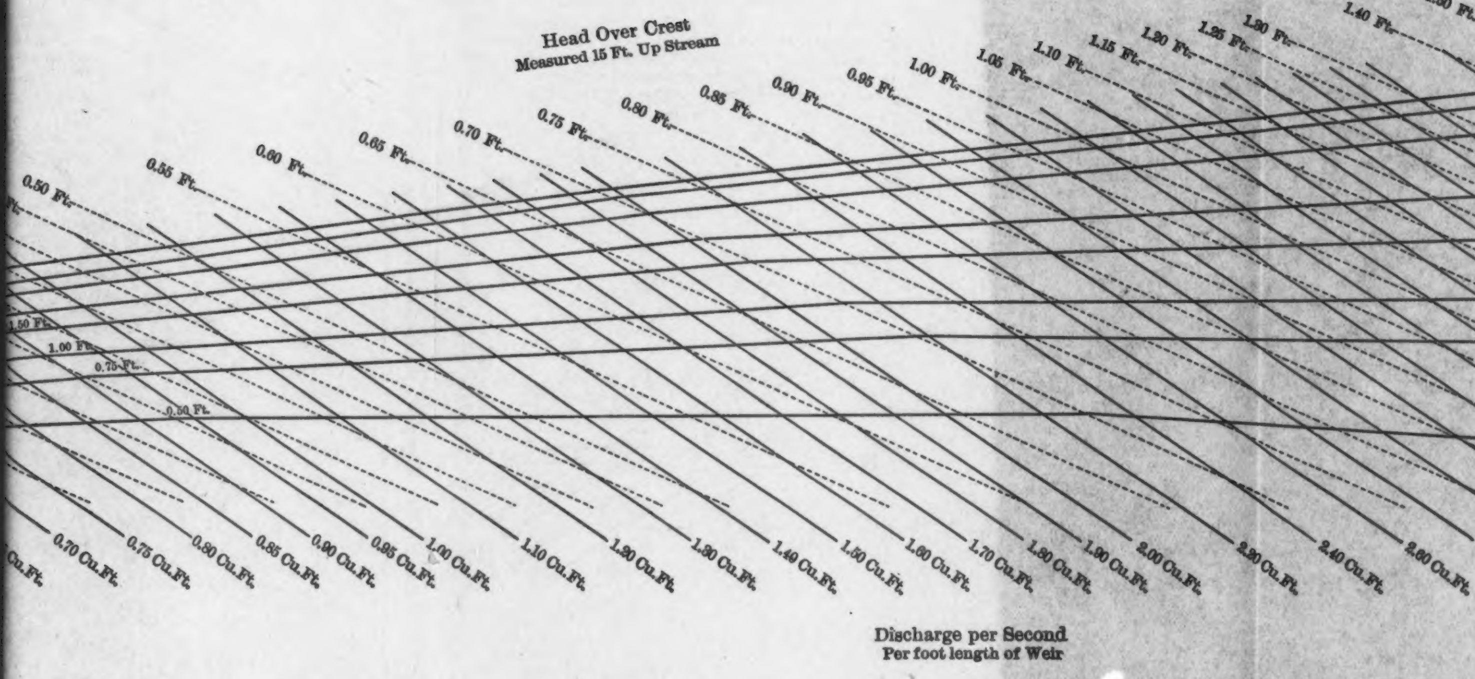

Head Over Crest
Measured 15 Ft. Up Stream
0.50 Ft.
0.55 Ft.
0.60 Ft.
0.65 Ft.
0.70 Ft.
0.75 Ft.
0.80 Ft.
0.85 Ft.
0.90 Ft.
0.95 Ft.
1.00 Ft.
1.05 Ft.
1.10 Ft.
1.15 Ft.
1.20 Ft.
1.25 Ft.
1.30 Ft.
1.40 Ft.
1.50 Ft.
1.50 Ft.
1.00 Ft.
0.75 Ft.
0.50 Ft.
Cu.Ft.
0.70 Cu.Ft.
0.75 Cu.Ft.
0.80 Cu.Ft.
0.85 Cu.Ft.
0.90 Cu.Ft.
0.95 Cu.Ft.
1.00 Cu.Ft.
1.10 Cu.Ft.
1.20 Cu.Ft.
1.30 Cu.Ft.
1.40 Cu.Ft.
1.50 Cu.Ft.
1.60 Cu.Ft.
1.70 Cu.Ft.
1.80 Cu.Ft.
1.90 Cu.Ft.
2.00 Cu.Ft.
2.20 Cu.Ft.
2.40 Cu.Ft.
2.60 Cu.Ft.
Discharge per Second
Per foot length of Weir

PLATE XXI.
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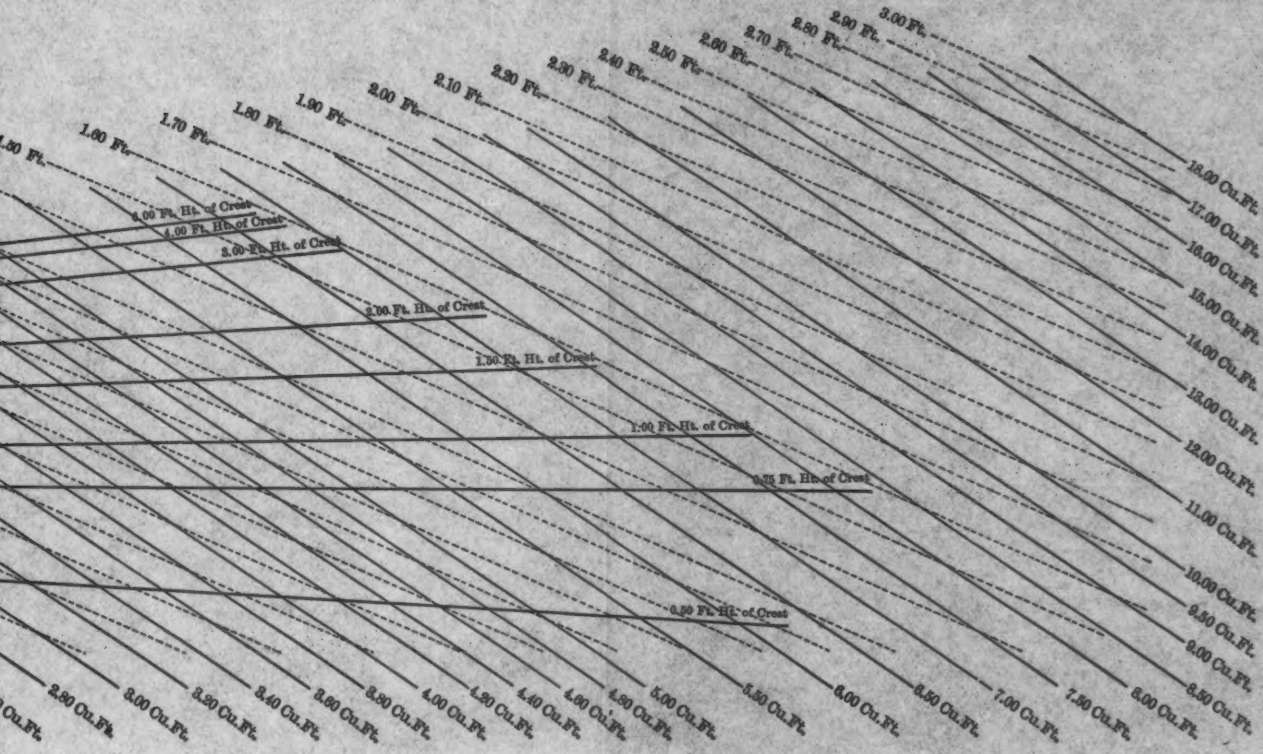

at a point slightly more than half way from the line representing a discharge of 4.60 cu. ft. per sec. to that representing a discharge of 4.80 cu. ft. per sec. The discharge per foot of length as read is 4.74 cu. ft. per sec. per ft. of length of weir, or 23.70 cu. ft. per sec. over the weir 5 ft. long.

Example 2.—What depth on the crest of a weir without end contractions will a flow of 8 cu. ft. per sec. reach if the weir is 2 ft. long and the height of the crest above the bottom of the channel of approach is 1.5 ft.?

The discharge per foot of length of weir is 4 cu. ft. per sec. The line on Plate XXI representing this discharge intersects that representing a weir of height 1.5 ft. between the lines representing the heads, 1.05 and 1.10 ft., at a point which shows that the head on the weir is 1.06 ft.

From a blue print of the original Plate XXI, which is about 50 in. long and 12 in. wide, all the discharges were taken for each and every run made by the original experimenters, and the difference, as a percentage, was computed between the quantities thus obtained and those originally measured. Each reading from the diagram was the average of two independent readings.

These results are shown on Plate XXII. Every run in every experiment is plotted. The abscissas represent the heads, in feet; the ordinates represent the quantity differences as percentages. A glance at the diagram shows that nearly all these results are below the + 1% line and above the — 1% line. This means that the diagram gives results in nearly all cases varying less than 1% from the original accurate data. How near the results given by the diagram fit the data obtained by any particular investigator in any series of runs can be seen by following the special line which represents that particular set of runs.

In the upper portion of Plate XXXIII it can be seen that, even with great care, an error of one-half of 1% in the discharge may be due to imperfections in measuring the head, yet the discharge diagram now being considered gives the accurate results just named, and therefore the assertion was made near the beginning of this paper that these results "can be relied on with greater confidence than can those obtained by using any one of the more or less complicated weir formulas which the experimenters themselves have devised."

b.—For High Heads.—Plate XXIII.—On Plate XXI the discharge curves are drawn only up to those heads actually used in the original experiments; but the curves may be extended for the highest weirs, so as to get approximate results for heads up to 3 ft. It is often desirable, however, to know the quantity flowing over weirs when the heads are much greater than those shown on Plate XXI.

Plate XXIII is made by using heads and the corresponding discharges given by the formulas of Bazin, Fteley and Stearns, and Francis.

Robert E. Horton, M. Am. Soc. C. E., "confidently" says* "that the Francis formula is applicable within 2 per cent. for heads as great as 5 feet, and by inference it is probably applicable for much greater heads as well." In the paper referred to, Mr. Horton gives his reasons for this assertion; and as there is at this time no more accurate or acceptable method of computing discharges over weirs for very high heads than to use these formulas, Plate XXIII has been prepared, and from it the results given by these formulas can be obtained.

For two reasons the velocity of approach, or the height of the weir, has been taken into account only in the computations made by the Bazin formula. First, because there is such uncertainty concerning the results these formulas give with very high heads, that making computations having apparently a high degree of accuracy is not justified. Secondly, because the Bazin formula provides a direct method of computing the discharge over weirs of various heights, and the Francis and Fteley and Stearns formulas both require going from one approximation to another several times in order to determine results that fit them accurately. The results obtained by the first-named formula were used largely in constructing Plate XXIII. The results given by the Francis formula and those obtained by using the Fteley and Stearns formula, without taking into account velocity of approach, are given on this plate also. These curves apply to cases where the velocity of approach is so slight that it may be neglected.

For these very high heads it is believed that Plate XXIII will give results as accurate as can be determined by any other method based on the experimental data at present available.

* Water-Supply and Irrigation Paper No. 150, page 40; issued by U. S. Geological Survey in 1906.

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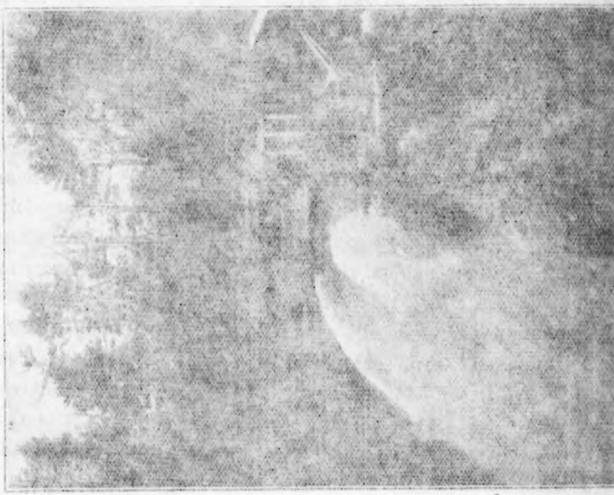
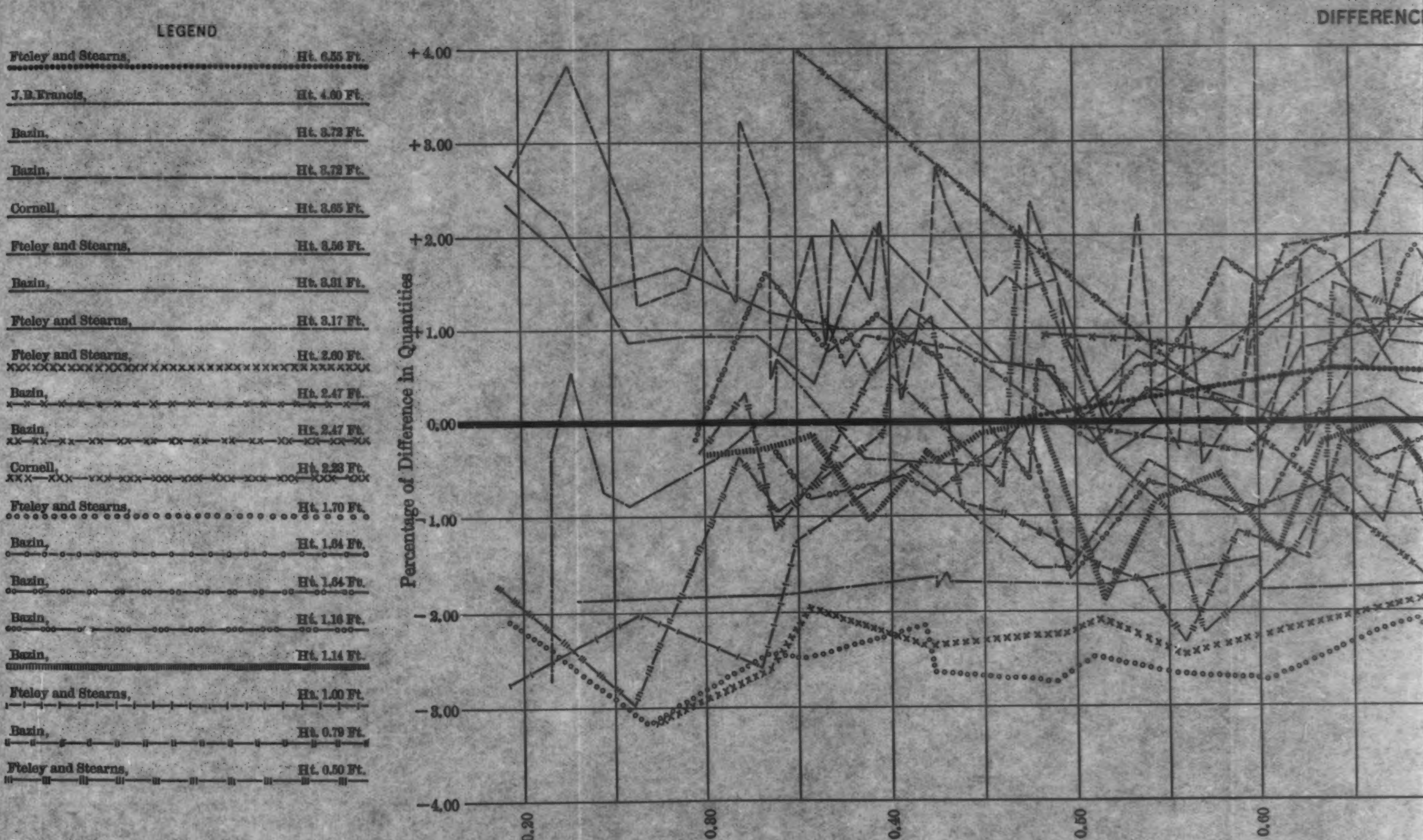
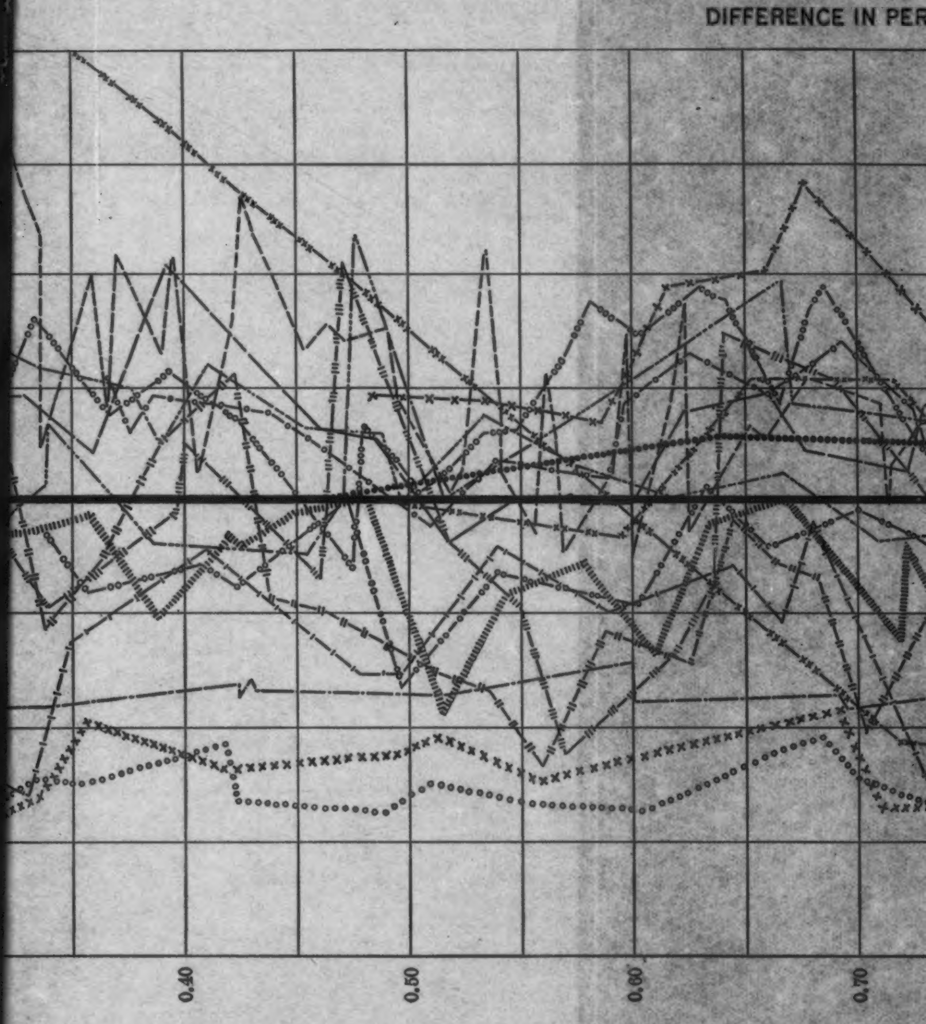
Fig. 3.—Lower End of Canal, Showing Lower Weir Under a High Head of Water.

Fig. 4.—Upper End of Canal, Showing Upper Weir Under a High Head of Water.

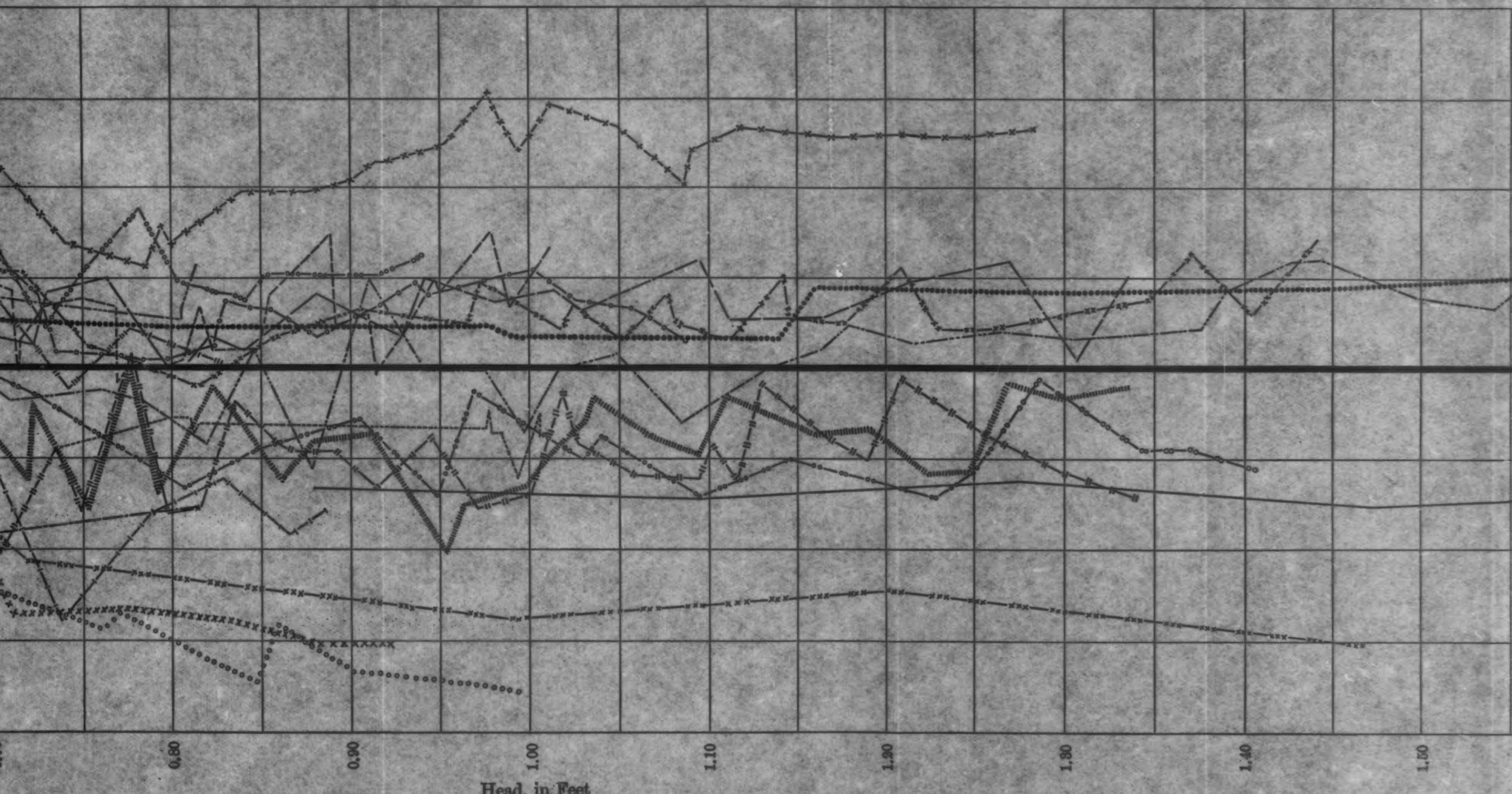


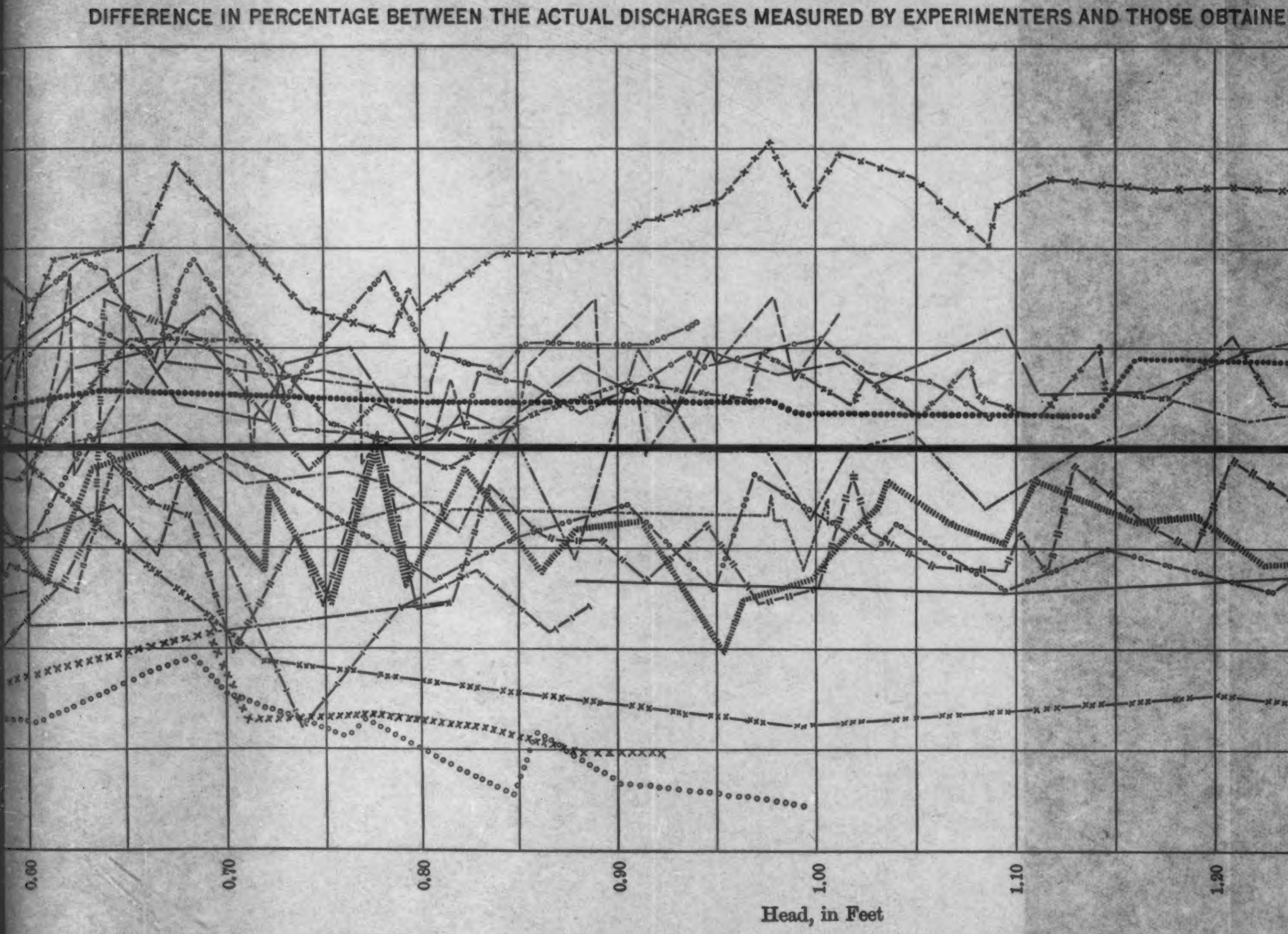


DIFFERENC
LEGEND
Fteley and Stearns, Ht. 6.55 Ft.
J.B.Francis, Ht. 4.60 Ft.
Bazin, Ht. 3.72 Ft.
Bazin, Ht. 3.72 Ft.
Cornell, Ht. 3.65 Ft.
Fteley and Stearns, Ht. 3.56 Ft.
Bazin, Ht. 3.31 Ft.
Fteley and Stearns, Ht. 3.17 Ft.
Fteley and Stearns, Ht. 2.60 Ft.
Bazin, Ht. 2.47 Ft.
Bazin, Ht. 2.47 Ft.
Cornell, Ht. 2.23 Ft.
Fteley and Stearns, Ht. 1.70 Ft.
Bazin, Ht. 1.64 Ft.
Bazin, Ht. 1.64 Ft.
Bazin, Ht. 1.16 Ft.
Bazin, Ht. 1.14 Ft.
Fteley and Stearns, Ht. 1.00 Ft.
Bazin, Ht. 0.79 Ft.
Fteley and Stearns, Ht. 0.50 Ft.
Percentage of Difference in Quantities
+4.00
+3.00
+2.00
+1.00
0.00
-1.00
-2.00
-3.00
-4.00
0.20
0.30
0.40
0.50
0.60


DIFFERENCE IN PER
0.40
0.50
0.60
0.70

N PERCENTAGE BETWEEN THE ACTUAL DISCHARGES MEASURED BY EXPERIMENTERS AND THOSE OBTAINED BY USE OF PLATE XXI





DIFFERENCE IN PERCENTAGE BETWEEN THE ACTUAL DISCHARGES MEASURED BY EXPERIMENTERS AND THOSE OBTAINE
0.60
0.70
0.80
0.90
1.00
1.10
1.20
Head, in Feet

PLATE XXII.
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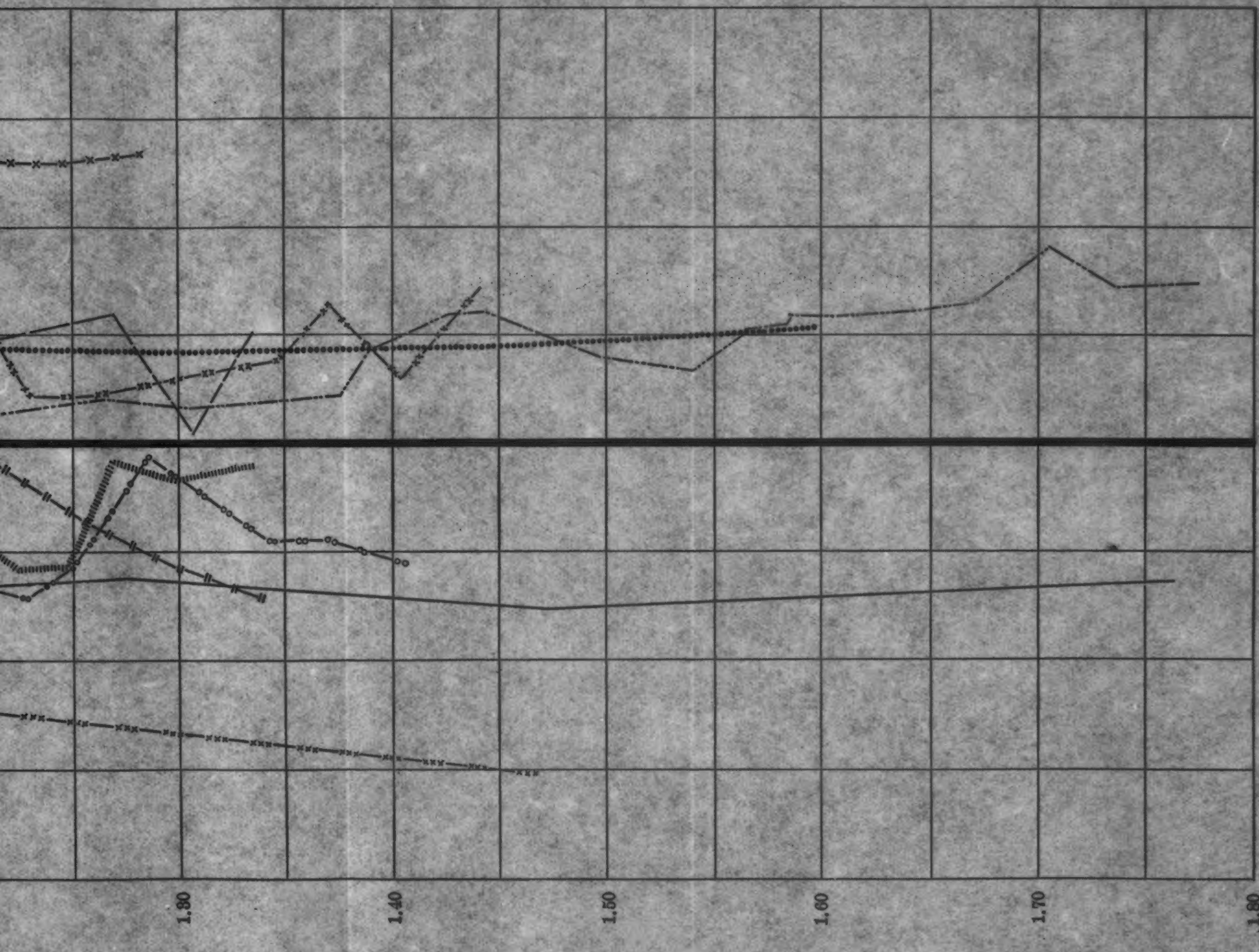
AINED BY USE OF PLATE XXI



That the diagrams here given are time-savers will be plain to the reader if he makes the computations which the formulas require in order to determine the quantities of discharge for weirs of the heights shown on Plate XXIII. If these results are furnished for a series of heads on each of these weirs, the writer will gladly put this additional information on the diagram.

(2).—Broad-Crested Weirs.—Plate XXIV.

(a).—Broad-Crested Weirs of One Height, Namely 11.25 Ft., the Height of the Model on Which the Experiments Were Made.—The curves on Plate XXIV, which give the discharge over a series of broad-crested weirs without end contractions, all having the same height, 11.25 ft., are based on a long and careful series of experiments in the hydraulic laboratory of Cornell University. Although the standard weir with which the water was measured, after it had flowed over the model, was not rated by actually catching and measuring the water volumetrically, the computations concerning the discharges measured over it* have been such that the results it gives can be accepted as accurate to within 2 or 3 per cent.†

On Plate XXIV also there are three series of lines. One gives the head or depth of water on the crest of the weir, measured 16 ft. up stream from the crest; another, the discharge per foot of length of weir; and the third, the width of the crest, in feet.

For every point on the diagram there is a definite discharge, and also a definite head. To ascertain the discharge for a given head on a weir of known width, find where the line representing the head intersects that representing the width. The discharge represented by this point of intersection is that for the given weir for 1 ft. of length.

It should be noted that, for comparatively low heads, all these broad-crested weirs give practically the same result for the same head.‡ At a particular head or point, however, for each width of crest, the quantity of discharge begins suddenly to increase much faster, in proportion to the head, than it has done before, and the line representing its discharge breaks away from *AA* and extends across the diagram in a

---

* "Weir Experiments, Coefficients, and Formulas," by Robert E. Horton, Water Supply Paper No. 150, U. S. Geological Survey, p. 39; also G. W. Rafter, "On the Flow of Water Over Dams." *Transactions*, Am. Soc. C. E., Vol. XLIV, p. 397.

† *Loc. cit.*, p. 40.

‡ The line, *AA*, represents this condition.

slanting direction to the line, *BB*, which represents the discharge over weirs with sharp crests. The head shown by the point at which such a line meets *BB* is that at which the sheet jumps over the crest without touching it. For all higher heads the sharp-crest condition obtains.

The condition that exists between the "broad-crest" or "open-channel" line and the "sharp-crest" line is perhaps produced by the tendency of the flowing stream to create a vacuum on top of the broad crest. This will be called the intermediate condition. Three examples of finding the discharge over a weir of width 1.5 ft. will be given: one for the "open-channel" condition, another for the "sharp-crest" condition, and a third for the intermediate condition.

Example 1.—(Open-Channel Condition.)—What is the discharge over a weir 1.5 ft. wide and 4 ft. long, where the head on the weir is 0.48 ft.?

The line representing a head of 0.48 ft. intersects that representing a weir of width 1.5 ft. at a point showing a discharge of 0.90 cu. ft. per sec. per ft. of length. The discharge over a weir 4 ft. long is four times this quantity, or 3.60 cu. ft. per sec.

Example 2.—(Sharp-Crest Condition.)—What is the discharge over a weir 1.5 ft. wide and 4 ft. long, where the head on the weir is 3.60 ft.?

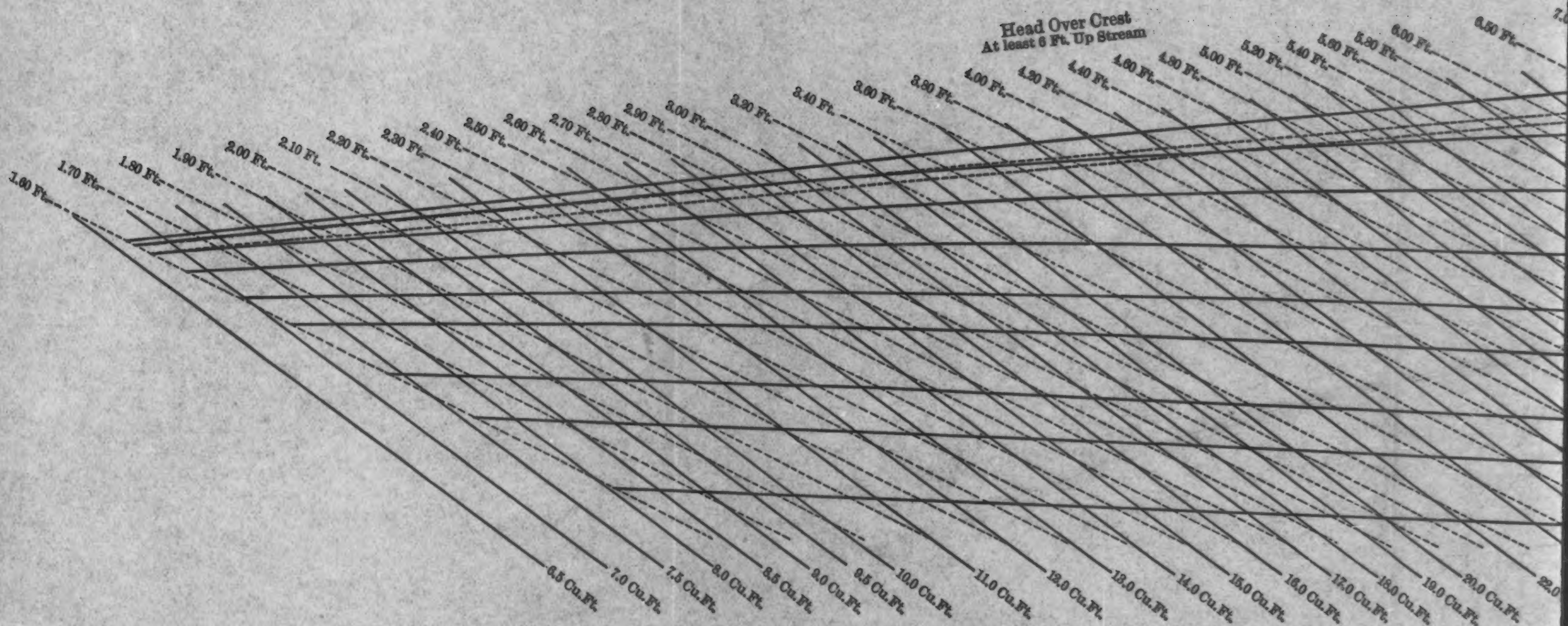
The line representing a head of 3.60 ft. intersects that representing a weir of width 1.5 ft. at a point showing a discharge of 23.5 cu. ft. per sec. per ft. of length. The discharge over the weir 4 ft. long is four times this quantity, or 94.0 cu. ft. per sec.

Example 3.—(Intermediate Condition.)—What is the discharge over a weir 1.5 ft. wide and 4 ft. long, where the head on the weir is 1.40 ft.?

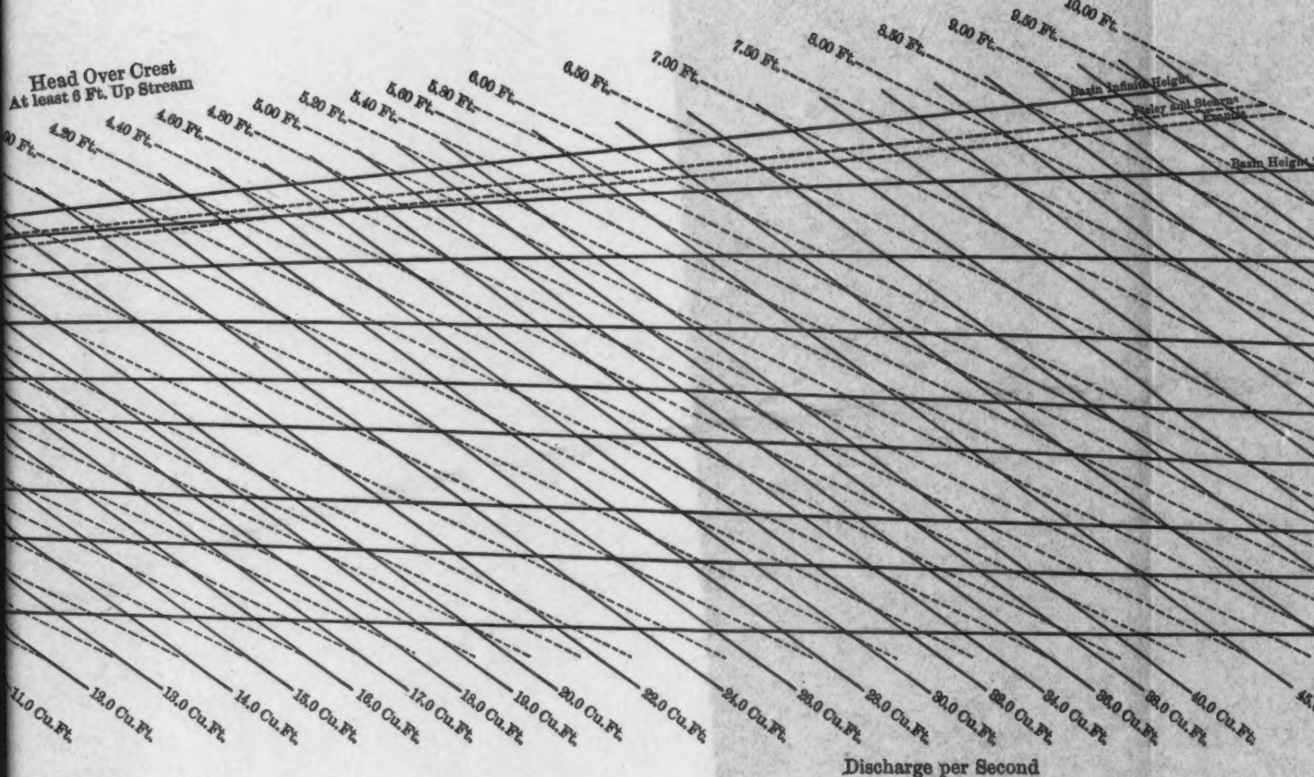
The line representing a head of 1.40 ft. intersects that representing a weir of width 1.5 ft. at a point showing a discharge of 4.93 cu. ft. per sec. per ft. of length. The discharge over the weir 4 ft. long is four times this quantity, or 19.72 cu. ft. per sec.

The dimensions and other details of the models on which these experiments were conducted are shown on Plate XXXI, and in Figs. 16, 17, and 20 to 27, inclusive.

(*b*).—*Broad-Crested Weirs of Any Height.*—It was at the suggestion of Gardner S. Williams, M. Am. Soc. C. E., that the writer prepared Table 0, showing the coefficients to be applied to the quantities of discharge given by Plate XXIV (and applicable only



Head Over Crest
At least 6 Ft. Up Stream
1.60 Ft.
1.70 Ft.
1.80 Ft.
1.90 Ft.
2.00 Ft.
2.10 Ft.
2.20 Ft.
2.30 Ft.
2.40 Ft.
2.50 Ft.
2.60 Ft.
2.70 Ft.
2.80 Ft.
2.90 Ft.
3.00 Ft.
3.20 Ft.
3.40 Ft.
3.60 Ft.
3.80 Ft.
4.00 Ft.
4.20 Ft.
4.40 Ft.
4.60 Ft.
4.80 Ft.
5.00 Ft.
5.20 Ft.
5.40 Ft.
5.60 Ft.
5.80 Ft.
6.00 Ft.
6.50 Ft.
6.5 Cu.Ft.
7.0 Cu.Ft.
7.5 Cu.Ft.
8.0 Cu.Ft.
8.5 Cu.Ft.
9.0 Cu.Ft.
9.5 Cu.Ft.
10.0 Cu.Ft.
11.0 Cu.Ft.
12.0 Cu.Ft.
13.0 Cu.Ft.
14.0 Cu.Ft.
15.0 Cu.Ft.
16.0 Cu.Ft.
17.0 Cu.Ft.
18.0 Cu.Ft.
19.0 Cu.Ft.
20.0 Cu.Ft.



Head Over Crest
At least 6 Ft. Up Stream
4.20 Ft.
4.40 Ft.
4.60 Ft.
4.80 Ft.
5.00 Ft.
5.20 Ft.
5.40 Ft.
5.60 Ft.
5.80 Ft.
6.00 Ft.
6.50 Ft.
7.00 Ft.
7.50 Ft.
8.00 Ft.
8.50 Ft.
9.00 Ft.
9.50 Ft.
10.00 Ft.
Bazin Infinite Height
Fteley and Stearns
Francis
Bazin Height
11.0 Cu.Ft.
12.0 Cu.Ft.
13.0 Cu.Ft.
14.0 Cu.Ft.
15.0 Cu.Ft.
16.0 Cu.Ft.
17.0 Cu.Ft.
18.0 Cu.Ft.
19.0 Cu.Ft.
20.0 Cu.Ft.
22.0 Cu.Ft.
24.0 Cu.Ft.
26.0 Cu.Ft.
28.0 Cu.Ft.
30.0 Cu.Ft.
32.0 Cu.Ft.
34.0 Cu.Ft.
36.0 Cu.Ft.
38.0 Cu.Ft.
40.0 Cu.Ft.
Discharge per Second
Per foot length of Weir

PLATE XXIII.
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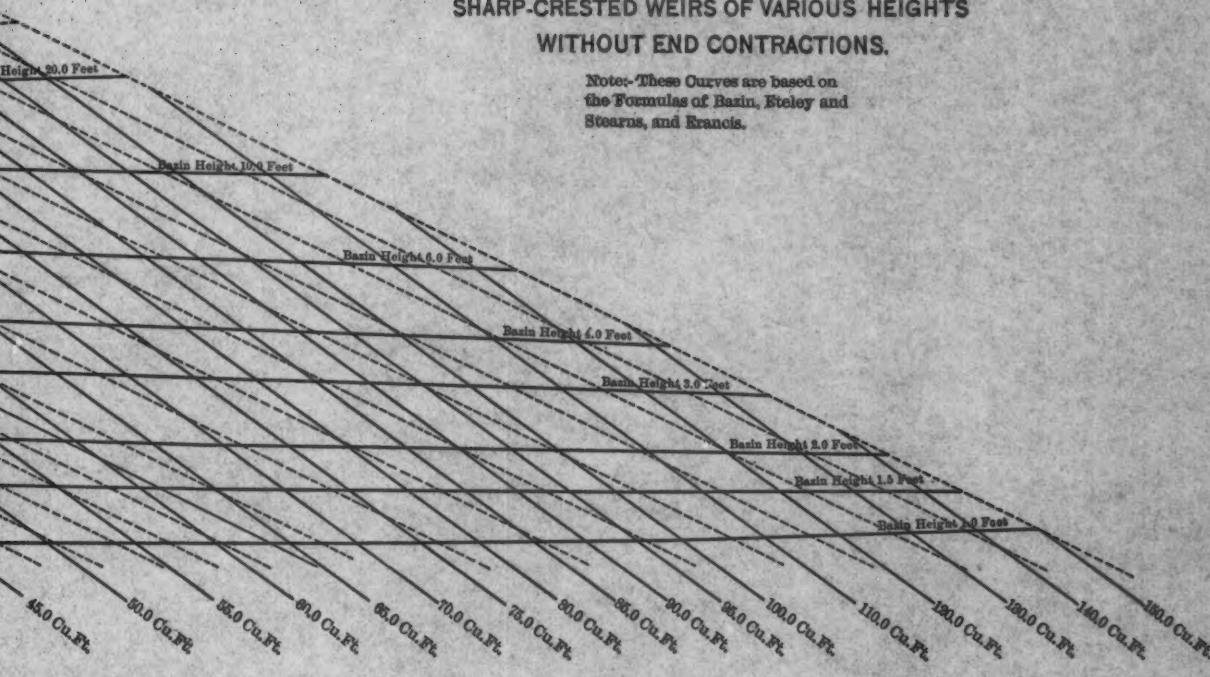
HEADS AND CORRESPONDING DISCHARGES OVER SHARP-CRESTED WEIRS OF VARIOUS HEIGHTS WITHOUT END CONTRACTIONS.

Note:- These Curves are based on the Formulas of Bazin, Eteley and Stearns, and Francis.



to weirs having the same height as the models on which the experiments were made, namely, 11.25 ft.), in order to obtain the discharge over broad-crested weirs of any height.

TABLE 0.—COEFFICIENTS BY WHICH THE QUANTITIES OF DISCHARGE OVER BROAD-CRESTED WEIRS AND IRREGULAR-CRESTED WEIRS OF 11.25 FT. (THE QUANTITIES GIVEN ON PLATES XXIV AND XXVI) ARE TO BE MULTIPLIED IN ORDER TO GIVE THE DISCHARGE OVER WEIRS OF THE HEIGHTS SHOWN IN THE TABLE AND FOR THE HEADS THERE GIVEN. COEFFICIENTS FOR OTHER HEADS AND FOR WEIRS OF OTHER HEIGHTS MAY BE FOUND BY INTERPOLATION.

Head, in feet.	Height of Weir, in Feet.										
	0.5	0.75	1.00	1.50	2.00	3.00	4.00	6.00	10.00	20.00	Infinite.
0.2	1.026	1.023	1.020	1.016	1.013	1.010	1.006	.....	.....	.....	.....
0.3	1.076	1.056	1.042	1.031	1.023	1.018	1.013		.....	.....	.....
0.4	1.107	1.077	1.063	1.041	1.035	1.024	1.018	1.012	.....	.....	.....
0.5	1.187	1.095	1.076	1.052	1.039	1.025	1.018	1.009	.....	.....	.....
0.6	1.160	1.102	1.086	1.053	1.040	1.024	1.014	1.008	.....	.....	.....
0.7	1.175	1.116	1.090	1.059	1.039	1.021	1.011	1.006	.....	.....	.....
0.8	1.217	1.137	1.105	1.073	1.058	1.027	1.015	1.010	.....	.....	.....
0.9	1.237	1.149	1.118	1.079	1.054	1.026	1.016	1.009	.....	.....	.....
1.0	1.265	1.166	1.130	1.088	1.063	1.028	1.016	1.009	.....	.....	.....
1.2	1.311	1.191	1.150	1.104	1.070	1.034	1.016	1.007	.....	.....	.....
1.4	......	1.216	1.171	1.117	1.083	1.035	1.018	1.011	.....	.....	.....
1.6	......	......	1.200	1.137	1.104	1.056	1.037	1.015	1.003	0.992	0.991
1.8	......	......	1.213	1.150	1.111	1.063	1.039	1.016	1.001	0.992	0.987
2.0	......	......	1.227	1.164	1.122	1.074	1.047	1.020	1.001	0.989	0.984
2.5	......	......	1.254	1.189	1.148	1.091	1.061	1.030	1.004	0.989	0.985
3.0	......	......	1.274	1.211	1.166	1.103	1.069	1.029	1.006	0.983	0.971
3.5	......	......	1.288	1.225	1.180	1.122	1.081	1.040	1.004	0.977	0.968
4.0	......	......	1.301	1.243	1.195	1.132	1.092	1.044	1.004	0.974	0.963
4.5	......	......	1.315	1.253	1.212	1.147	1.104	1.055	1.009	0.978	0.957
5.0	......	......	1.318	1.266	1.220	1.156	1.116	1.060	1.007	0.971	0.950
5.5	......	......	1.320	1.270	1.227	1.168	1.118	1.067	1.011	0.972	0.948
6.0	......	......	1.321	1.269	1.230	1.171	1.127	1.068	1.009	0.965	0.937
6.5	......	......	1.312	1.264	1.224	1.166	1.123	1.068	1.008	0.960	0.926
7.0	......	......	1.317	1.270	1.232	1.171	1.127	1.072	1.011	0.961	0.922
7.5	......	......	1.314	1.273	1.236	1.177	1.136	1.075	1.013	0.958	0.917
8.0	......	......	1.313	1.270	1.233	1.177	1.134	1.078	1.014	0.954	0.911
8.5	......	......	1.315	1.270	1.236	1.180	1.136	1.078	1.012	0.951	0.908
9.0	......	......	1.311	1.270	1.239	1.187	1.141	1.084	1.013	0.951	0.899
9.5	......	......	1.306	1.269	1.238	1.188	1.145	1.084	1.014	0.948	0.901
10.0	......	......	1.299	1.264	1.234	1.182	1.143	1.082	1.013	0.944	0.892

These coefficients were obtained by assuming that the effect of the height of the weir in the case of broad crests is the same as in that of sharp crests. The discharges over sharp-crested weirs having the heights given in Table 0 and the heads there indicated, were found from Plates XXI and XXIII for sharp-crested weirs. The ratios were then found between the quantities thus obtained and the discharge

over a sharp-crested weir of height 11.25 ft. for the corresponding heads. These ratios are the coefficients given in Table 0. Although the degree of accuracy they give is necessarily uncertain, it is believed that the results found by their use will be the best obtainable until additional reliable data are secured.

(3).—Weirs with Irregular Cross-Sections.—Plates XXV and XXVI.

(*a*).—*Specific Weirs of Heights Shown.*—Plates XXV and XXVI show the actual results obtained by experiments on a series of models of irregular cross-sections in the hydraulic laboratory of Cornell University. There is not sufficient similarity in the models used to make a general diagram from the results obtained, which will apply to weirs with any cross-section, or even to those with somewhat similar cross-sections, as was done in the case of the sharp-crested weirs on Plate XXI or the broad-crested weirs on Plate XXIV.

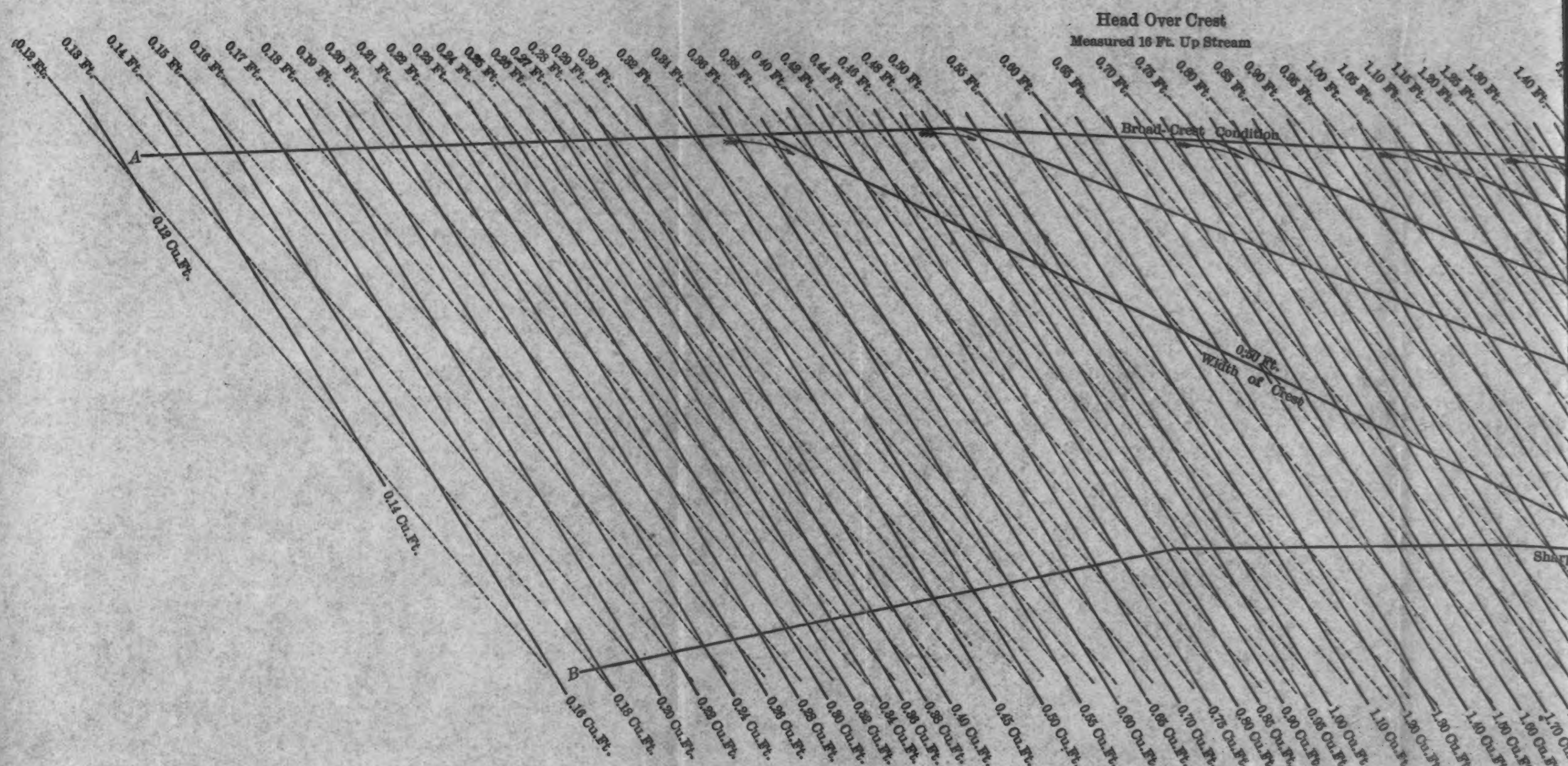
It is believed, however, that from these two diagrams a close estimate can be made of the quantity of water flowing over almost any weir or dam of irregular cross-section. As readings are taken on these plates exactly as they are on Plates XXI and XXIV, no further examples or explanations need be given on this point.

Cross-sections of these models, showing their dimensions and method of construction are given on these two plates, and in Figs. 17 to 19, and 28 to 45, inclusive. The curves are also marked with the model number, so that the line for any particular form may be easily followed.

(*b*).—*Weirs of Any Height, if Similar to Those Models Shown on Plate XXVI That Have a Height of 11.25 Ft.*—For the same reason that the coefficients in Table 0 may be applied to the discharge over broad-crested weirs, having a height of 11.25 ft., in order to obtain the discharge over similar weirs of different heights, these same coefficients may be applied to the discharges obtained from Plate XXVI for the models having a height of 11.25 ft., in order to obtain the discharge over similar models of different heights. The models having a height of 11.25 ft. from which experiments were actually made are the following: XXX, XXXIII, XXXIV, XXXV, XXXVI, XXXVIII, and XXXIX. For further explanation, see "(*b*) Broad-Crested Weirs of Any Height."

Fig. 5.—Model of Chambly Dam, in Upper End of Canal, Cornell Hydraulic Laboratory.

Fig. 6.—Model of Plattsburg Dam, in Upper End of Canal, Cornell Hydraulic Laboratory.


Head Over Crest
Measured 16 Ft. Up Stream
0.12 Ft.
0.13 Ft.
0.14 Ft.
0.15 Ft.
0.16 Ft.
0.17 Ft.
0.18 Ft.
0.19 Ft.
0.20 Ft.
0.21 Ft.
0.22 Ft.
0.23 Ft.
0.24 Ft.
0.25 Ft.
0.26 Ft.
0.27 Ft.
0.28 Ft.
0.29 Ft.
0.30 Ft.
0.32 Ft.
0.34 Ft.
0.36 Ft.
0.38 Ft.
0.40 Ft.
0.42 Ft.
0.44 Ft.
0.46 Ft.
0.48 Ft.
0.50 Ft.
0.55 Ft.
0.60 Ft.
0.65 Ft.
0.70 Ft.
0.75 Ft.
0.80 Ft.
0.85 Ft.
0.90 Ft.
0.95 Ft.
1.00 Ft.
1.05 Ft.
1.10 Ft.
1.15 Ft.
1.20 Ft.
1.25 Ft.
1.30 Ft.
1.40 Ft.
Broad-Crest Condition
A
0.12 Cu.Ft.
0.50 Ft.
Width of Crest
0.14 Cu.Ft.
Sharp
B
0.16 Cu.Ft.
0.18 Cu.Ft.
0.20 Cu.Ft.
0.22 Cu.Ft.
0.24 Cu.Ft.
0.26 Cu.Ft.
0.28 Cu.Ft.
0.30 Cu.Ft.
0.32 Cu.Ft.
0.34 Cu.Ft.
0.36 Cu.Ft.
0.38 Cu.Ft.
0.40 Cu.Ft.
0.45 Cu.Ft.
0.50 Cu.Ft.
0.55 Cu.Ft.
0.60 Cu.Ft.
0.65 Cu.Ft.
0.70 Cu.Ft.
0.75 Cu.Ft.
0.80 Cu.Ft.
0.85 Cu.Ft.
0.90 Cu.Ft.
0.95 Cu.Ft.
1.00 Cu.Ft.
1.10 Cu.Ft.
1.20 Cu.Ft.
1.30 Cu.Ft.
1.40 Cu.Ft.
1.50 Cu.Ft.
1.60 Cu.Ft.
1.70 Cu.Ft.

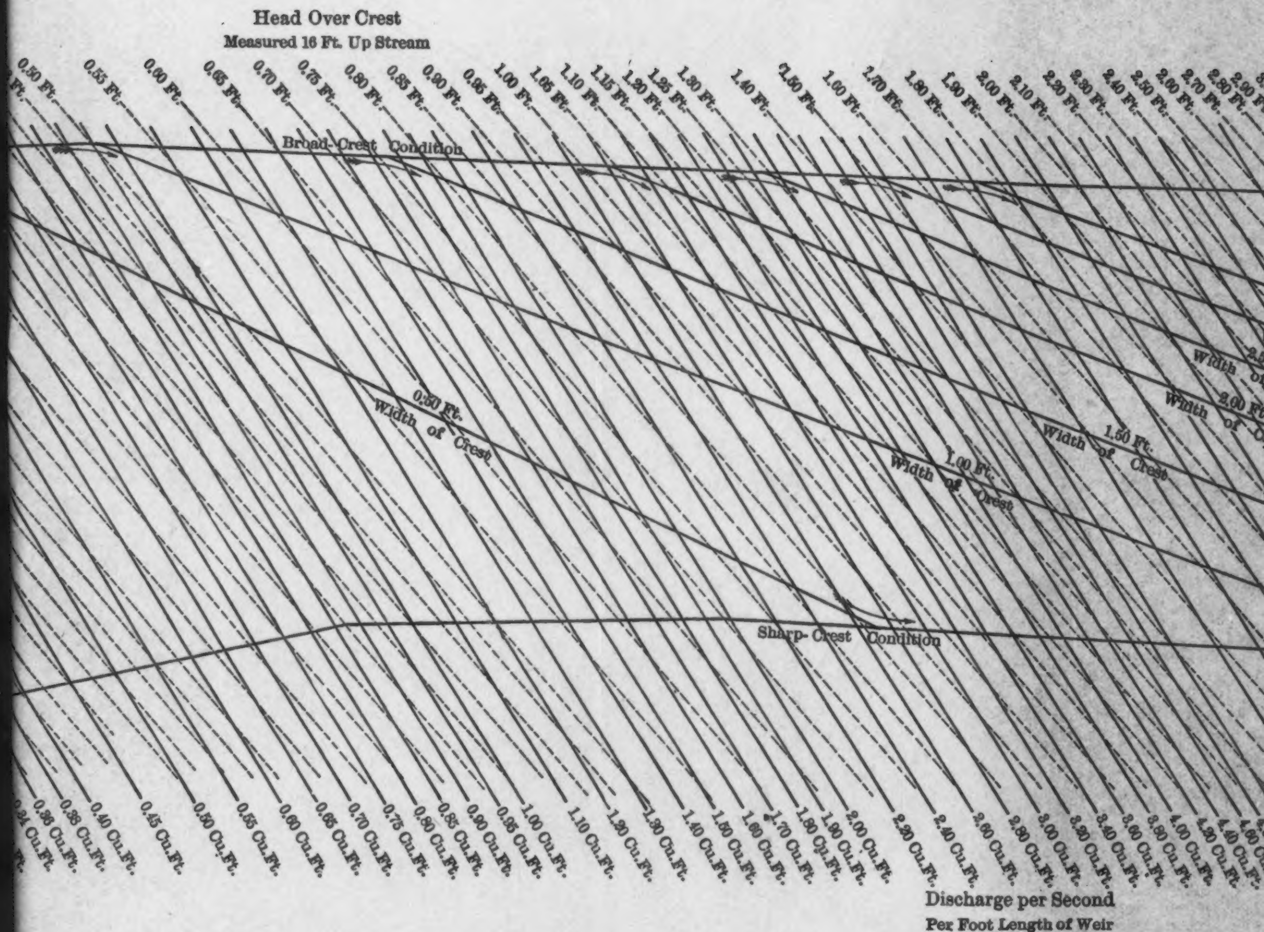

Head Over Crest
Measured 16 Ft. Up Stream
Broad-Crest Condition
0.50 Ft. Width of Crest
1.00 Ft. Width of Crest
1.50 Ft. Width of Crest
2.00 Ft. Width of Crest
Sharp-Crest Condition
Discharge per Second
Per Foot Length of Weir

PLATE XXIV.
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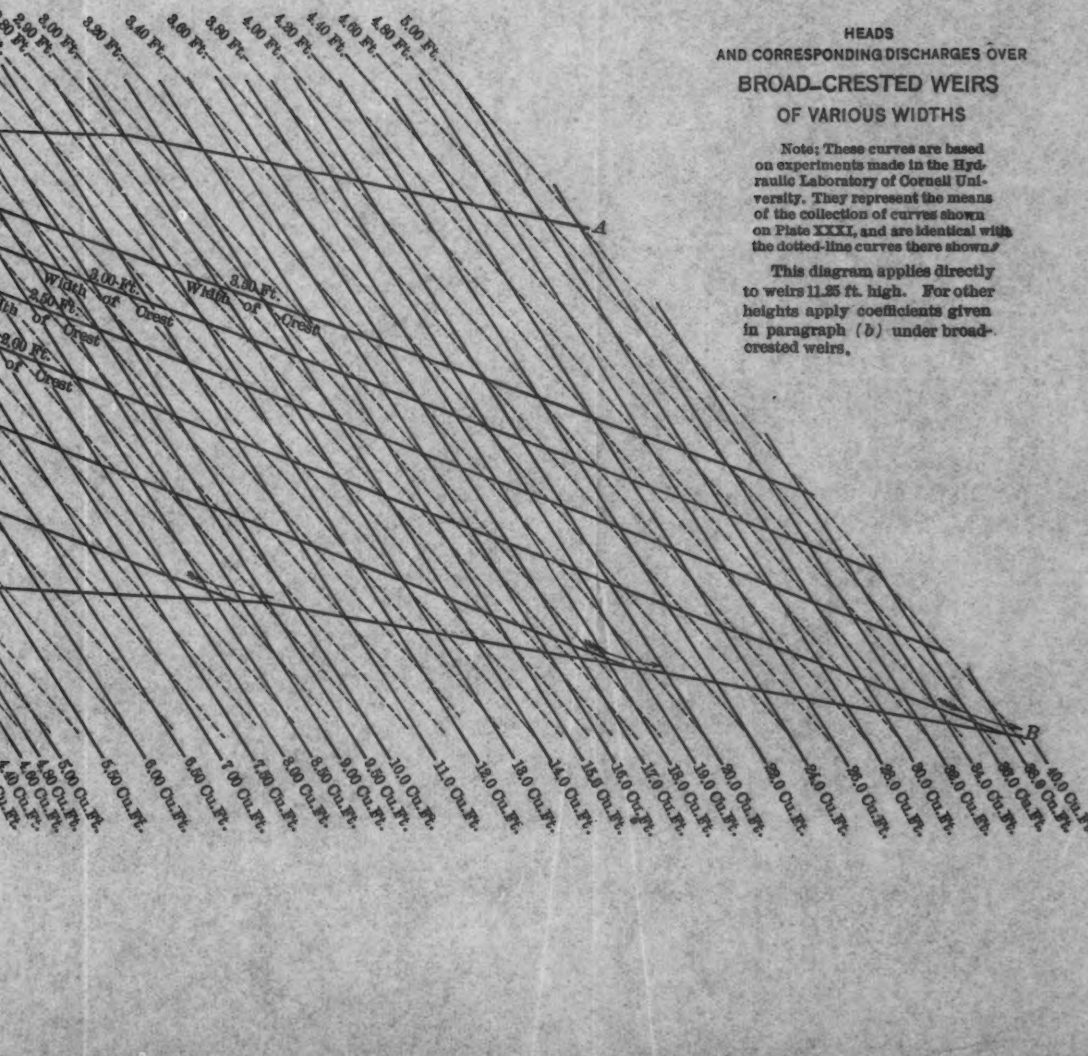
HEADS
AND CORRESPONDING DISCHARGES OVER

BROAD-CRESTED WEIRS

OF VARIOUS WIDTHS

Note: These curves are based on experiments made in the Hydraulic Laboratory of Cornell University. They represent the means of the collection of curves shown on Plate XXXI, and are identical with the dotted-line curves there shown.

This diagram applies directly to weirs 11.25 ft. high. For other heights apply coefficients given in paragraph (b) under broad-crested weirs.



C.—Formulas for Calculating the Discharge Over Weirs.

(1).—The Francis Formula.

The Francis formula for weirs without end contractions is used most generally. It is as follows:

$$Q = 3.33\ H^{\frac{3}{2}} \text{*}$$

in which

Q = discharge, in cubic feet per second per foot of length of weir;

and H = effective head of water on the weir, in feet; its value to be obtained by using the formula:

$$H = (h + h')^{\frac{3}{2}} - h'^{\frac{3}{2}} \dagger$$

in which

h = the measured head, or the vertical distance, in feet, between the horizontal plane containing the crest of the weir and the plane surface of the water in the channel of approach;

and $h' = \dfrac{V^2}{2g}$, V being the velocity of the water in the channel of approach, and g the acceleration of gravity.

Mr. Francis has pointed out that making this correction for the velocity of approach "will be requisite only when great precision is required." For, he says, in one of his experiments, the water in the channel of approach had a mean velocity of about 1 ft. per sec., and this had the effect of increasing the discharge about 2%, and, in another experiment, the velocity was about 0.5 ft. per sec., and this had the effect of increasing the discharge about 1 per cent. "These examples," writes Mr. Francis, "will enable the operator to judge, in each case, of the necessity of going through the troublesome calculation for correcting the depth on the weir."‡

(2).—Formula of Fteley and Stearns.

For weirs without end contractions, Messrs. Fteley and Stearns, from their experiments, deduced the following formula:

$$Q = 3.31\ (h + 1.5\ h')^{\frac{3}{2}} + 0.007\ \S$$

in which

Q = discharge, in cubic feet per second per foot of length of weir;

---

* "Lowell Hydraulic Experiments," by J. B. Francis, pp. 131 and 133.

† *Loc. cit.*, p. 117.

‡ *Loc. cit.*, p. 135.

§ *Transactions*, Am. Soc. C. E., Vol. XII, pp. 11 and 82.

h = measured head, or the vertical distance, in feet, between the horizontal plane containing the crest of the weir and the plane surface of the water in the channel of approach;

and $h' = \frac{V^2}{2\,g}$, V being the mean velocity of the water in the channel of approach, and g the acceleration of gravity.

(3).—The Bazin Formula.

Bazin's is the only weir formula that has been proposed, up to this time, which, for accurate results, does not necessitate taking into account the velocity of the water in the channel of approach. His formula, in English units, is as follows:

$$Q = \frac{2}{3}\left(0.6075 + \frac{0.0148}{h}\right)\left[1 + 0.55\left(\frac{h}{p+h}\right)^2\right] h\sqrt{2\,gh}$$

in which

Q = discharge, in cubic feet per second per foot of length of weir;

h = the measured head, or the vertical distance, in feet, between the horizontal plane containing the crest of the weir and the plane surface of the water in the channel of approach;

p = the vertical distance between the bottom of the channel of approach and the horizontal plane containing the weir's crest;

g = the acceleration of gravity, in feet per second per second.

(4).—Formulas Proposed by the Writer.

Equations of the form, $Q = m\,h^n$, are derived in this paper for all the weirs of all the forms herein considered. The great variation in the values of m and n shown on Plates XXVII, XXX, and XXXII, and in Tables C, D, E, and F, are strong arguments against the use of only one formula for computing the discharge over weirs of various sizes, especially when the quantity of water to be measured varies through wide limits.

Again, by the methods explained in this paper, diagrams can be prepared that give the same results as the formulas of the form, $Q = m\,h^n$. By the diagrams herein presented, the discharges over nearly all well-known experimental weirs can be found. Based on the results given by these experiments, other diagrams are presented which indicate with accuracy the discharge over weirs of almost all forms and sizes.

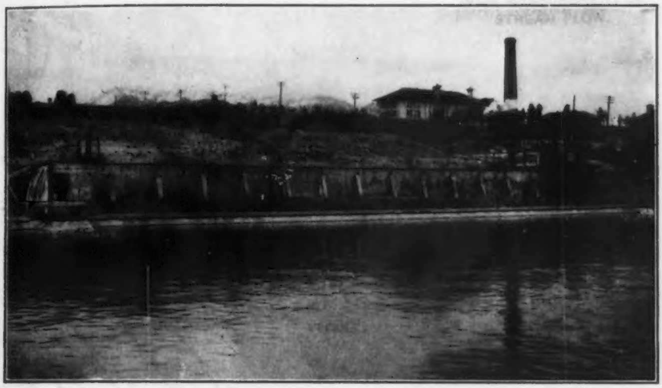


FIG. 7.—GENERAL VIEW OF HYDRAULIC LABORATORY OF UNIVERSITY OF UTAH AND THIRTEENTH EAST STREET RESERVOIR.

FIG. 8.—MEASURING BASIN, HYDRAULIC LABORATORY OF UNIVERSITY OF UTAH, AND THIRTEENTH EAST STREET RESERVOIR.

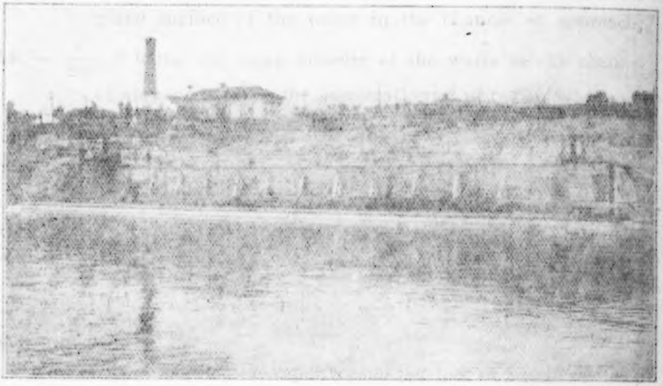


FIG. 7.—GENERAL VIEW OF HYDRAULIC LABORATORY OF UNIVERSITY OF UTAH AND THIRTEENTH EAST STREET RESERVOIR.

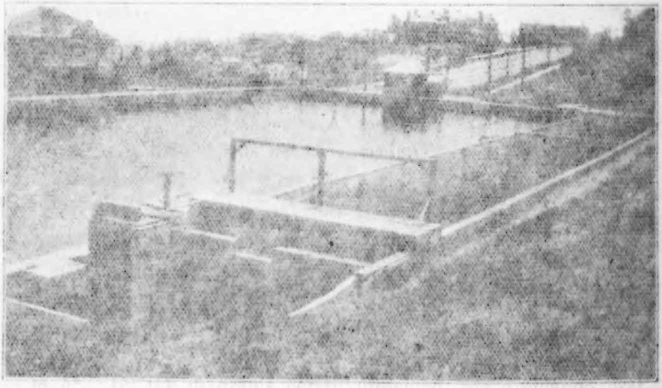
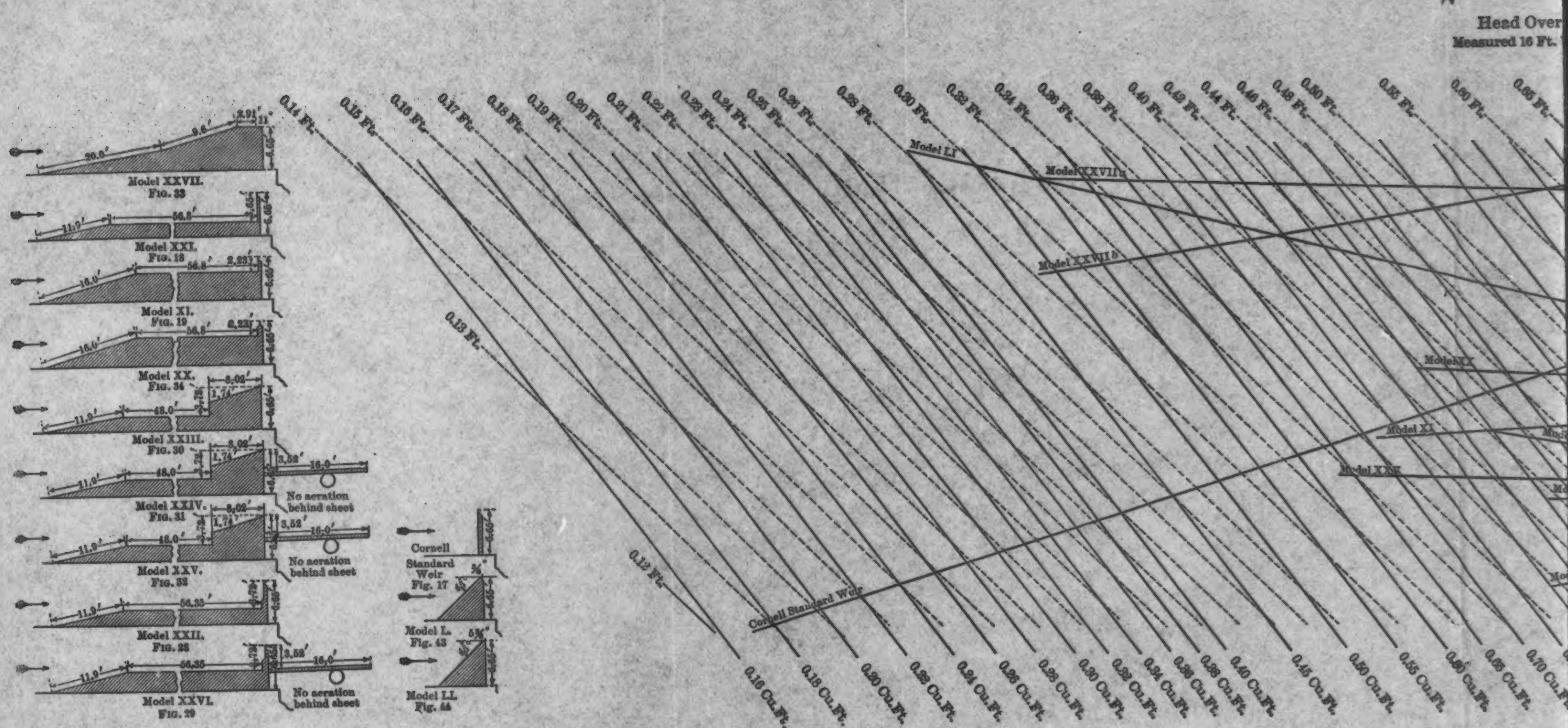


FIG. 8.—MEASURING BASIN, HYDRAULIC LABORATORY OF UNIVERSITY OF UTAH, AND THIRTEENTH EAST STREET RESERVOIR.


Head Over
Measured 16 Ft.
0.12 Ft.
0.13 Ft.
0.14 Ft.
0.15 Ft.
0.16 Ft.
0.17 Ft.
0.18 Ft.
0.19 Ft.
0.20 Ft.
0.21 Ft.
0.22 Ft.
0.23 Ft.
0.24 Ft.
0.25 Ft.
0.26 Ft.
0.28 Ft.
0.30 Ft.
0.32 Ft.
0.34 Ft.
0.36 Ft.
0.38 Ft.
0.40 Ft.
0.42 Ft.
0.44 Ft.
0.46 Ft.
0.48 Ft.
0.50 Ft.
0.55 Ft.
0.60 Ft.
0.65 Ft.
Model LI
Model XXVII a
Model XXVII b
Model XX
Model XI
Model XXI
Cornell Standard Weir
0.16 Cu. Ft.
0.18 Cu. Ft.
0.20 Cu. Ft.
0.22 Cu. Ft.
0.24 Cu. Ft.
0.26 Cu. Ft.
0.28 Cu. Ft.
0.30 Cu. Ft.
0.32 Cu. Ft.
0.34 Cu. Ft.
0.36 Cu. Ft.
0.38 Cu. Ft.
0.40 Cu. Ft.
0.45 Cu. Ft.
0.50 Cu. Ft.
0.55 Cu. Ft.
0.60 Cu. Ft.
0.65 Cu. Ft.
0.70 Cu. Ft.
Model XXVII. Fig. 33
Model XXI. Fig. 18
Model XI. Fig. 19
Model XX. Fig. 34
Model XXIII. Fig. 30
Model XXIV. Fig. 31
No aeration behind sheet
Model XXV. Fig. 32
No aeration behind sheet
Model XXII. Fig. 28
Model XXVI. Fig. 29
No aeration behind sheet
Cornell Standard Weir Fig. 17
Model L. Fig. 43
Model LI. Fig. 44

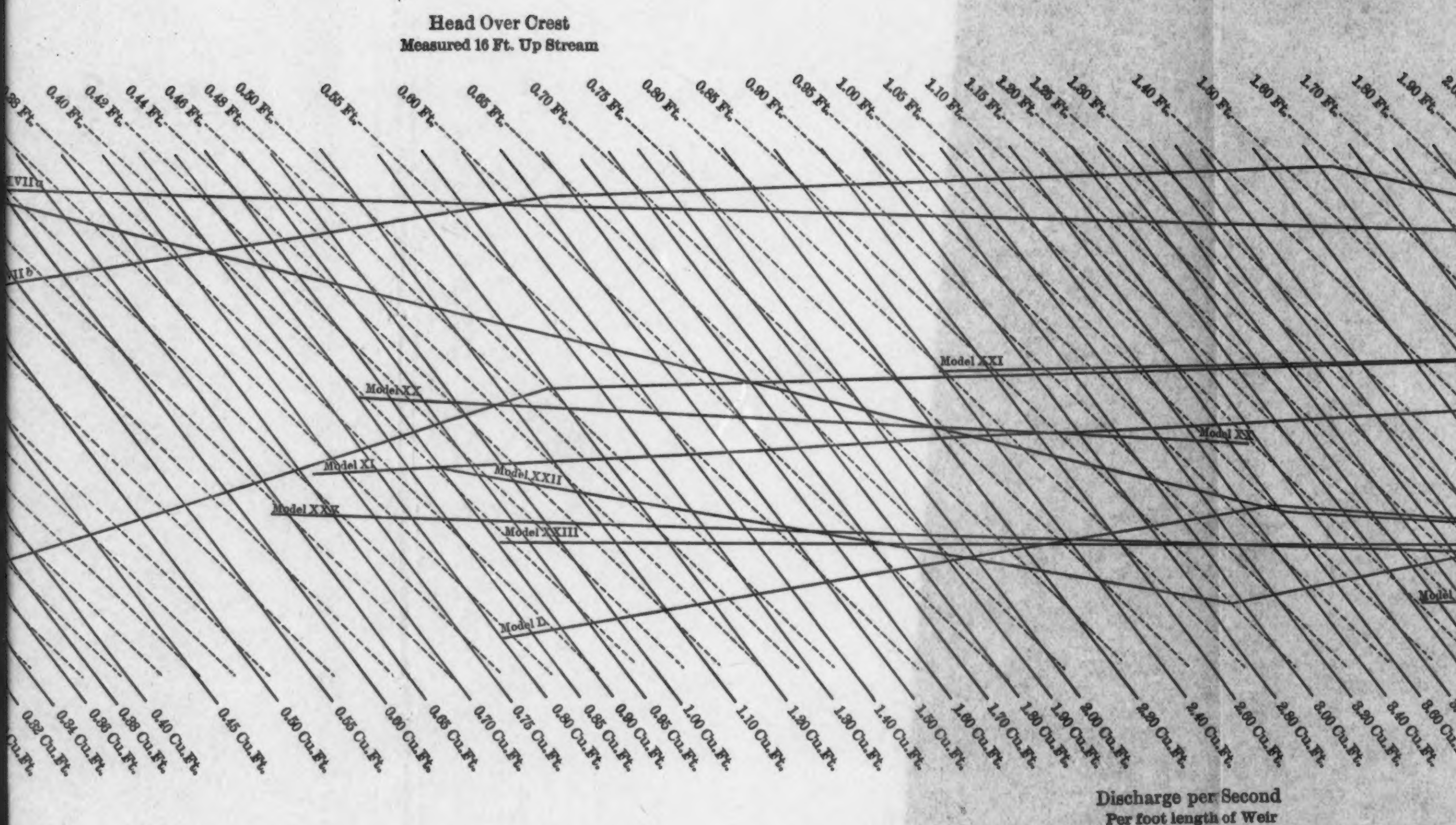

Head Over Crest
Measured 16 Ft. Up Stream
Model XXI
Model XX
Model XI
Model XXII
Model XXX
Model XXIII
Model D.
Discharge per Second
Per foot length of Weir

PLATE XXV.
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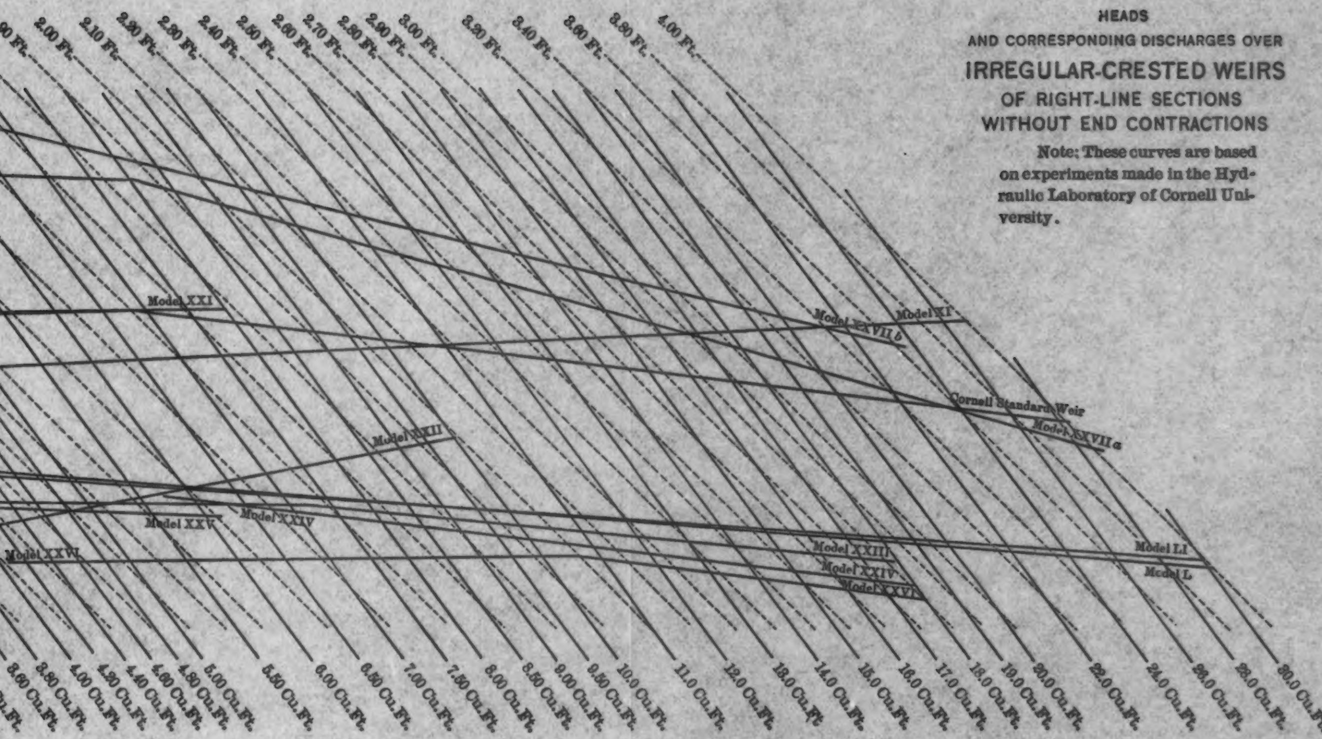
HEADS
AND CORRESPONDING DISCHARGES OVER
IRREGULAR-CRESTED WEIRS
OF RIGHT-LINE SECTIONS
WITHOUT END CONTRACTIONS

Note: These curves are based on experiments made in the Hydraulic Laboratory of Cornell University.



This formula, $Q = m\ h^n$, may be written

$$\log. Q = n \log. h + \log. m.$$

If the logarithms of heads of less than 1 ft. be expressed as negative quantities, the Thacher rule may be used for computing the values of $n \log. h$. If, to the results thus obtained, the constant quantity, $\log. m$, be added, the quantities obtained are the logarithms of the discharges desired.

By this method, computations of quantities of discharge by any formula of the form, $Q = m\ h^n$, are made as easily and quickly as by the Francis formula, $Q = 3.33\ H^{\frac{3}{2}}$.

For finding the discharge over a weir, the diagram has two advantages over the formula: First, it gives results without computation; secondly, results obtained by the diagram do not appear to contain an accuracy which is not warranted.

D.—Comparative Accuracy of the Formulas.

The Francis formula has been used more than any other, for the reason, perhaps, that engineers, generally, have constructed weirs so as to feel satisfied, justly or otherwise, with the results given by this formula, without making a correction for the velocity of approach. By doing so they have avoided what Francis himself has very properly called "the troublesome calculation for correcting the depth on the weir."*

In the right-hand portion of Fig. 8, Plate XXVII*a*, the points located symmetrically with respect to the line having the equation, $Q = m\ h^{1.5032}$ (in which $\log. m = 0.5257$, or $m = 3.355$), show how accurately this equation fits the Francis experiments when they have not been corrected for velocity of approach. Two broken lines are shown passing through the points in the left-hand portion of this drawing. That marked "Francis Formula" shows where these points should be if the equation fitted them perfectly. Those points to which the velocity of approach has not been applied are to the right of this line at a distance which indicates that, for a head of 1.0 ft., the formula gives a computed discharge 0.9% too small (see scale on Plate XXVII*a*). The error is slightly less for lower heads. When to these experiments "the troublesome calculation for velocity of approach" has been applied, the Francis formula, for a head of 1.0 ft., still gives a result in error 0.2%, as the diagram shows, and the equa-

* "Lowell Hydraulic Experiments," p. 135.

tion, $Q = m\ h^{1.4964}$ (in which log. $m = 0.5214$, or $m = 3.322$), fits these experiments almost perfectly.

A comparison of the values given in Table 8A, and in Table XIII of the "Lowell Hydraulic Experiments," shows that the computed or corrected heads in these Francis experiments do not agree. This lack of agreement is due to the fact that Francis based his corrections on the velocity of approach 6 ft. up stream from the crest, and the writer's are based on a velocity of approach computed from the cross-section of the channel just up stream from the crest of the weir. These results would agree if the bottom of the channel of approach were level, or, in other words, if the cross-section of the channel of approach were constant.

Note what effect this small difference in cross-section makes, and then imagine the uncertainty that necessarily comes into the results by measuring water with a weir above which sediment and gravel accumulate until at times the depth of the channel of approach decreases to zero. In practice, such a condition often prevails.

It may be well to draw attention at this point to the accuracy with which equations of the form, $Q = m\ h^n$, can be made to fit experiments by the methods herein presented, and also to the clearness with which the degree of accuracy attained can be seen.

It will be noted that, of these two broken lines, that on the left fits the results better than that on the right. The latter represents the Francis formula, and the former is the line that best fits the Francis experiments. For a head of 1 ft. there is a difference in quantity of only 0.17% (by the scale on Plate XXVII*a*) between the results of the Francis formula and those of the "best fitting line", yet this small difference is shown clearly and accurately on the diagram. The broken line in the extreme right-hand side of the figure shows the results obtained by using the final diagram for a weir of this height. For a head of 1 ft. it gives quantities about 0.7% less than the Francis results not corrected for velocity of approach, and quantities about 1.8% less than the Francis results when these are thus corrected. It is the influence of other experiments that draws this line so far to the right.

E.—Method of Determining the Proposed Formulas.

No exhaustive study of the quantities discharged over a weir for known heads is required to discover that one equation of the form,

Fig. 9.—Laboratory Buildings, Canals, Diverting Device, and Waste-way of Hydraulic Laboratory of University of Utah.

Fig. 10.—Receiving Basin, Canal No. 1, Weir No. 1, Canal No. 2, and Weir No. 2 of Hydraulic Laboratory of University of Utah.

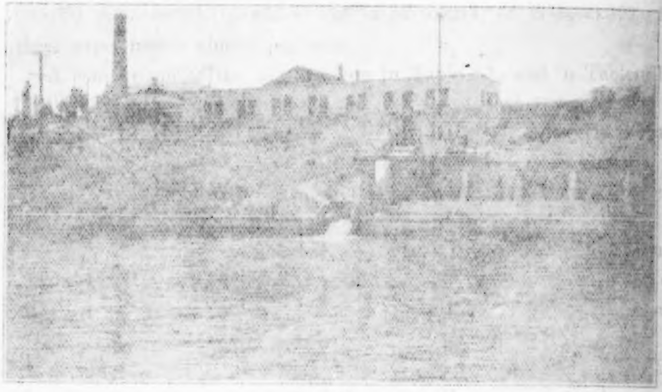
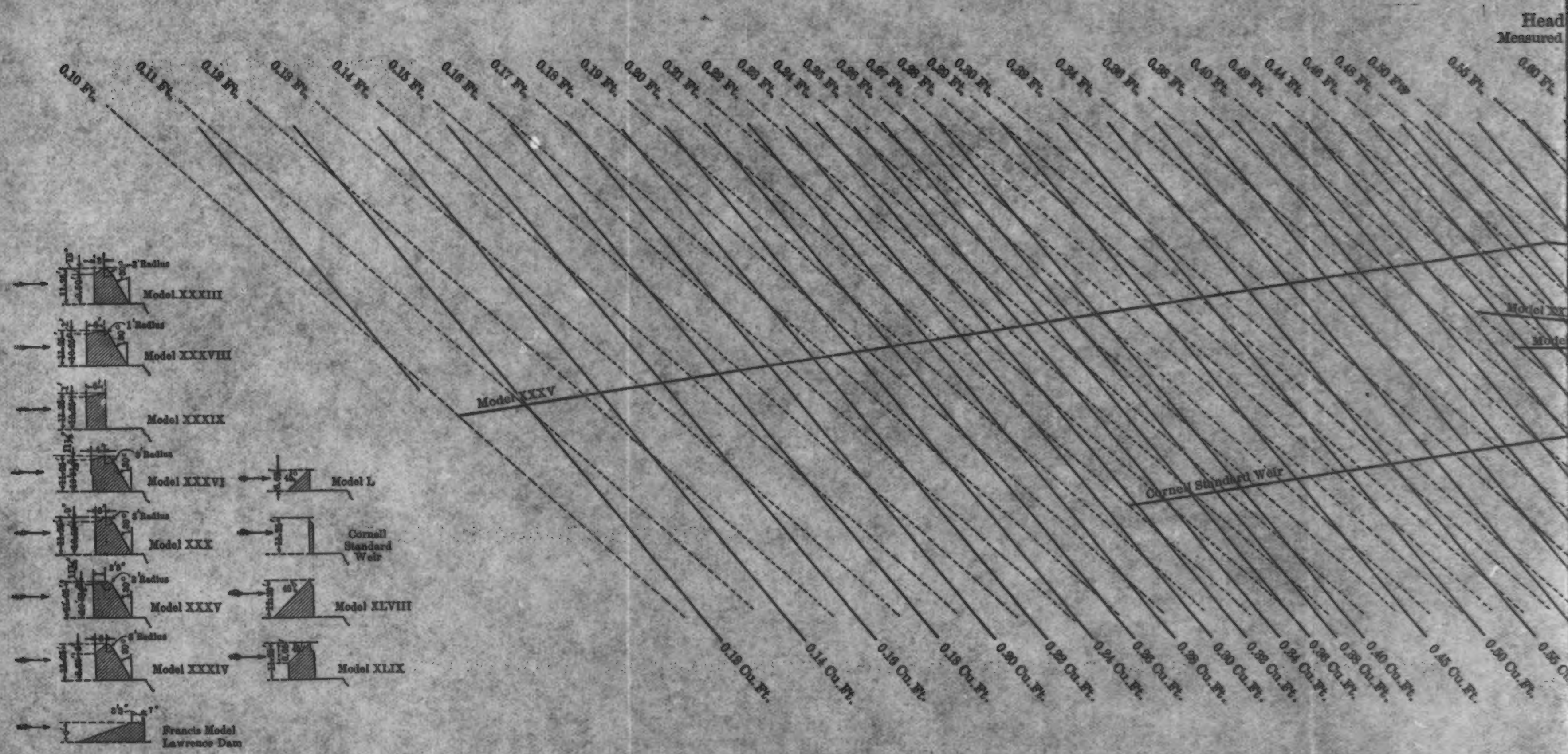
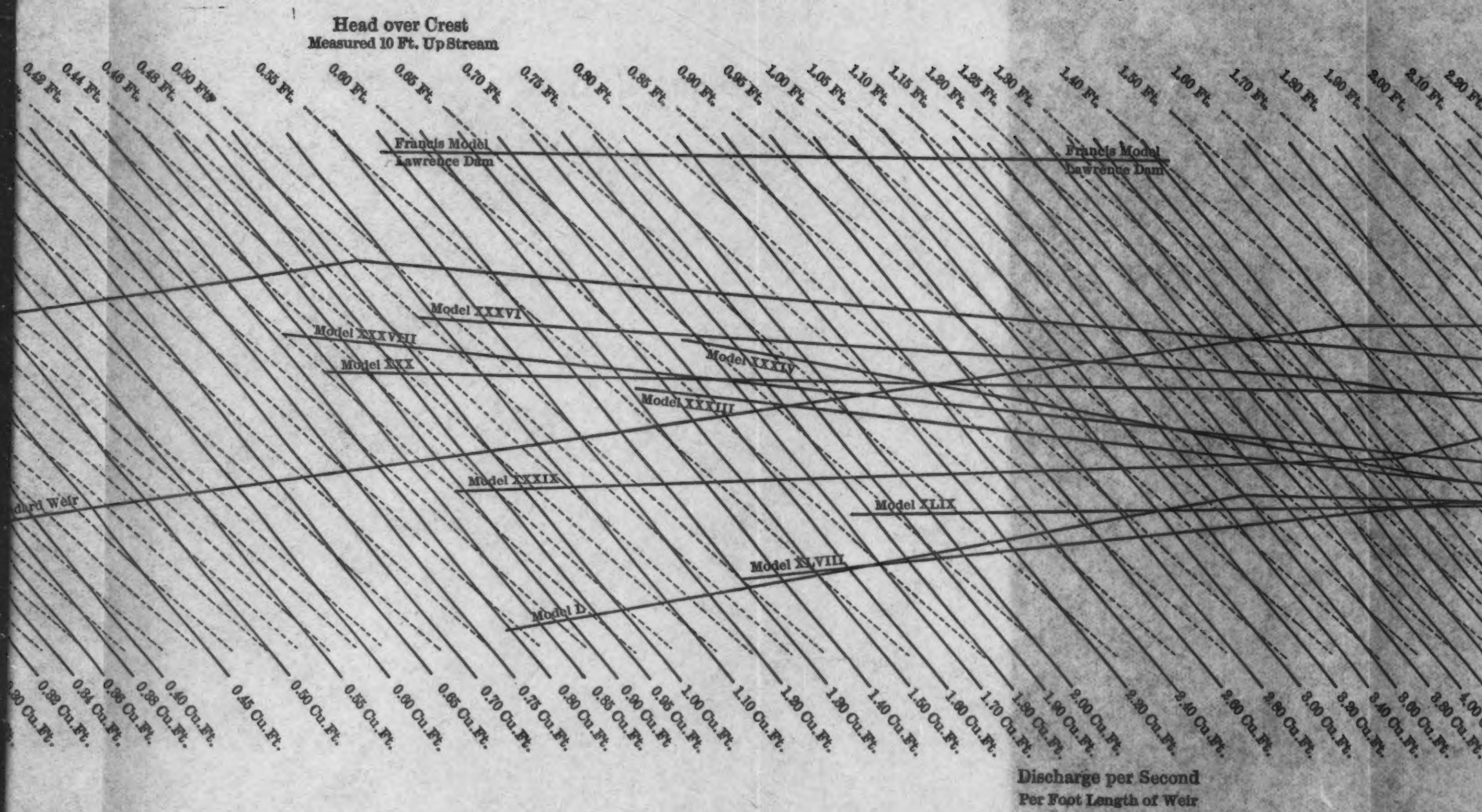


Fig. 9.—Laboratory Buildings, Canals, Diverting Device, and Waste-way of Hydraulic Laboratory of University of Utah.

Fig. 10.—Receiving Basin, Canal No. 1, Weir No. 1, Canal No. 2, and Weir No. 2 of Hydraulic Laboratory of University of Utah.


Head
Measured
0.10 Ft.
0.11 Ft.
0.12 Ft.
0.13 Ft.
0.14 Ft.
0.15 Ft.
0.16 Ft.
0.17 Ft.
0.18 Ft.
0.19 Ft.
0.20 Ft.
0.21 Ft.
0.22 Ft.
0.23 Ft.
0.24 Ft.
0.25 Ft.
0.26 Ft.
0.27 Ft.
0.28 Ft.
0.34 Ft.
0.36 Ft.
0.38 Ft.
0.40 Ft.
0.42 Ft.
0.44 Ft.
0.46 Ft.
0.48 Ft.
0.55 Ft.
0.60 Ft.
Model XXXIII
Model XXXVIII
Model XXXIX
Model XXXVI
Model L
Model XXX
Cornell Standard Weir
Model XXXV
Model XLVIII
Model XXXIV
Model XLIX
Francis Model Lawrence Dam
Model XXXV
Cornell Standard Weir
0.12 Cu. Ft.
0.14 Cu. Ft.
0.16 Cu. Ft.
0.18 Cu. Ft.
0.20 Cu. Ft.
0.22 Cu. Ft.
0.24 Cu. Ft.
0.26 Cu. Ft.
0.28 Cu. Ft.
0.30 Cu. Ft.
0.32 Cu. Ft.
0.34 Cu. Ft.
0.36 Cu. Ft.
0.38 Cu. Ft.
0.40 Cu. Ft.
0.45 Cu. Ft.
0.50 Cu. Ft.



Head over Crest
Measured 10 Ft. Up Stream
0.42 Ft.
0.44 Ft.
0.46 Ft.
0.48 Ft.
0.50 Ft.
0.55 Ft.
0.60 Ft.
0.65 Ft.
0.70 Ft.
0.75 Ft.
0.80 Ft.
0.85 Ft.
0.90 Ft.
0.95 Ft.
1.00 Ft.
1.05 Ft.
1.10 Ft.
1.15 Ft.
1.20 Ft.
1.25 Ft.
1.30 Ft.
1.40 Ft.
1.50 Ft.
1.60 Ft.
1.70 Ft.
1.80 Ft.
1.90 Ft.
2.00 Ft.
2.10 Ft.
2.20 Ft.
Francis Model
Lawrence Dam
Francis Model
Lawrence Dam
Model XXXVI
Model XXXVIII
Model XXX
Model XXXIV
Model XXXIII
Model XXXIX
Model XLIX
Model XLVIII
Model D
0.32 Cu. Ft.
0.34 Cu. Ft.
0.36 Cu. Ft.
0.38 Cu. Ft.
0.40 Cu. Ft.
0.45 Cu. Ft.
0.50 Cu. Ft.
0.55 Cu. Ft.
0.60 Cu. Ft.
0.65 Cu. Ft.
0.70 Cu. Ft.
0.75 Cu. Ft.
0.80 Cu. Ft.
0.85 Cu. Ft.
0.90 Cu. Ft.
0.95 Cu. Ft.
1.00 Cu. Ft.
1.10 Cu. Ft.
1.20 Cu. Ft.
1.30 Cu. Ft.
1.40 Cu. Ft.
1.50 Cu. Ft.
1.60 Cu. Ft.
1.70 Cu. Ft.
1.80 Cu. Ft.
1.90 Cu. Ft.
2.00 Cu. Ft.
2.20 Cu. Ft.
2.40 Cu. Ft.
2.60 Cu. Ft.
2.80 Cu. Ft.
3.00 Cu. Ft.
3.20 Cu. Ft.
3.40 Cu. Ft.
3.60 Cu. Ft.
3.80 Cu. Ft.
Discharge per Second
Per Foot Length of Weir

PLATE XXVI.
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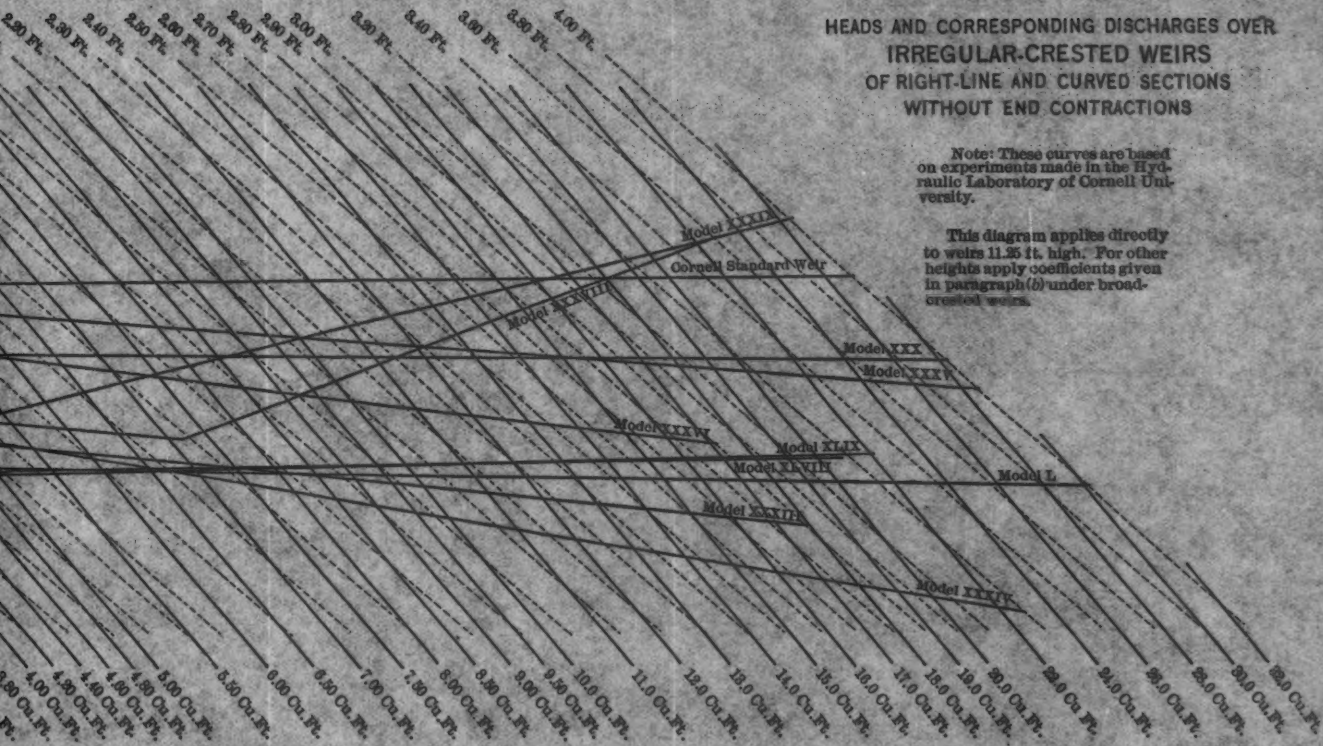
HEADS AND CORRESPONDING DISCHARGES OVER IRREGULAR-CRESTED WEIRS OF RIGHT-LINE AND CURVED SECTIONS WITHOUT END CONTRACTIONS

Note: These curves are based on experiments made in the Hydraulic Laboratory of Cornell University.

This diagram applies directly to weirs 11.25 ft. high. For other heights apply coefficients given in paragraph (b) under broad-crested weirs.



$Q = m\,h^n$, with constant values for m and n will not give the discharge with a high degree of accuracy if the values of h cover a wide range. Nevertheless, when the values of h cover but a limited range, the highest degree of accuracy that the experimental data warrant can be obtained with an equation of this form.

Here, as elsewhere in this paper, wherever Q is used, it designates the quantity of water discharged over a weir, in cubic feet per second per foot of length of weir; h is used to designate the observed head of water on a weir, in feet, unless otherwise plainly stated. (This h is not measured in still water.)

With given values of h and the corresponding values of Q, efforts have been made to determine equations of the form named that satisfy these values.

First, plotting on logarithmic paper was tried, but even when using paper with a 20-in. base, the curvature of the lines determined by the plotted points was so slight that the best fitting lines that could be obtained were more likely, when their equations were applied to the given data, to be ill- than to be well-fitting.

Secondly, the logarithms of the Q's and the h's, on a scale as large as could be used with any degree of convenience, were plotted; but the results were little if any more satisfactory than those obtained from the plotting on logarithmic paper.

Thirdly, the method of finding the centers of gravity of the upper and lower halves of the curves that result from plotting the logarithms and passing a line analytically through the points thus found, was used. This method gives results somewhat more satisfactory than the others, when applied to the determination of a single line or equation for the whole set of experiments; but if one equation does not give satisfactory results, this method gives little or no indication of the number of equations necessary, or the point at which divisions should be made, before attempting to find the two or more equations required. This method of investigation had been applied faithfully to nearly all the experiments herein presented, and gave results which were considered very satisfactory, before the final method was discovered.

Fourthly, by the final method of treating these experiments, lines having equations which give results with a degree of accuracy as high as the experiments warrant can be passed through the points plotted.

By this method the trial equation, $Q = m\, h^{1.5000}$ (log. $m = 0.5000$), is first applied. The differences between the logarithms of the quantities measured and the logarithms of the quantities computed by this trial formula, are plotted; the logarithms of the heads are used as ordinates, and the differences just mentioned as abscissas. The scale on which the differences are plotted is ten times that on which the heads are plotted, so that the angles and curves of the various lines are greatly exaggerated; with little difficulty, therefore, the number of equations needed to fit the experiments is selected, and the lines representing these equations are easily and readily drawn.

Taking the logarithms of both sides of the general equation,

$$Q = m\, h^n,$$

gives log. $Q = n$ log. $h +$ log. m.

This is the equation of a straight line the intercept of which on the Y or log. Q axis (the line having the equation, log. $h = 0$) is log. m; n is the slope of the line, or the tangent of the angle the line makes with the X or log. h axis (the line having the equation, log. $Q = 0$).

Let YY be the log. Q axis, and let XX be the log. h axis (as shown by Fig. A, at the right of Plate XXVIIIb). The log. h axis is put in the vertical position because it is usual to measure and represent heads in that direction.

The locus of the trial formula, or equation, log. $Q = 1.5000$ log. $h + 0.5000$, is the line, ab. The point, a, on the log. Q axis is found by substituting in the foregoing equation log. $h = 0$, and the point, b, on the line, log. $h = -1$, is found by substituting in that equation log. $h = -1$. The distance, Oa, or the "intercept on the Y axis", is 0.5000 in this particular equation, or it is log. m in the general equation. The value of n is (from the equation of a straight line) the tangent of the angle the line makes with the log. h axis, that is, the tangent of the angle, Oca. In this case it is

$$\frac{Oa}{Oc} = \frac{ad}{db} = \frac{1.5}{1.0} = 1.5.$$

Lines actually representing the equations which fit the data considered vary but slightly from the line, ab. Hence, the differences in the values of log. Q cannot be shown with sufficient accuracy on a drawing of reasonable size when rectangular axes are used. The line, ab, therefore, is made vertical on the same horizontal lines, and from

FIG. 11.—DIVERTING DEVICE AND BAFFLES IN SOUTH END OF MEASURING BASIN, HYDRAULIC LABORATORY OF UNIVERSITY OF UTAH.

FIG. 12.—WEIR NO. 3, DIVERTING DEVICE, AND WASTE-WAY OF HYDRAULIC LABORATORY OF UNIVERSITY OF UTAH.

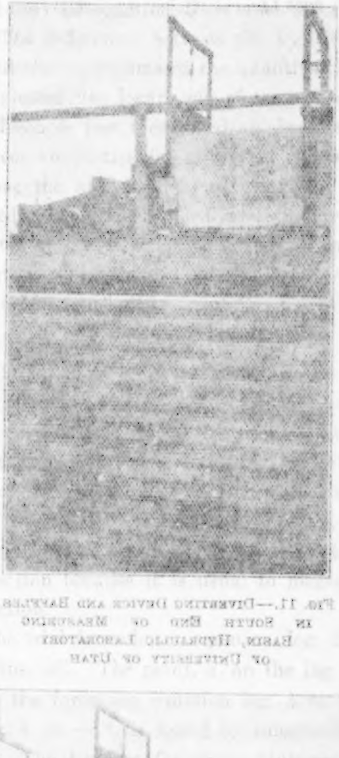


FIG. 11.—DIVERTING DEVICE AND BAFFLES IN SOUTH END OF MEASURING BASIN, HYDRAULIC LABORATORY OF UNIVERSITY OF UTAH.

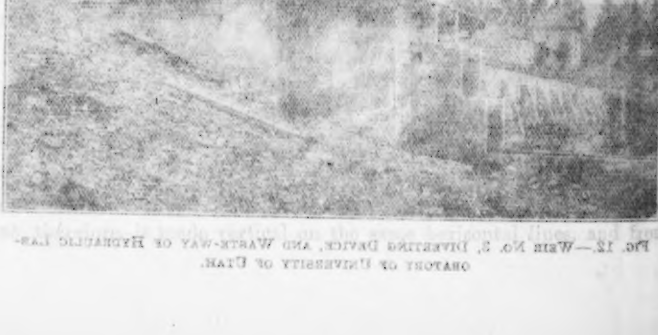
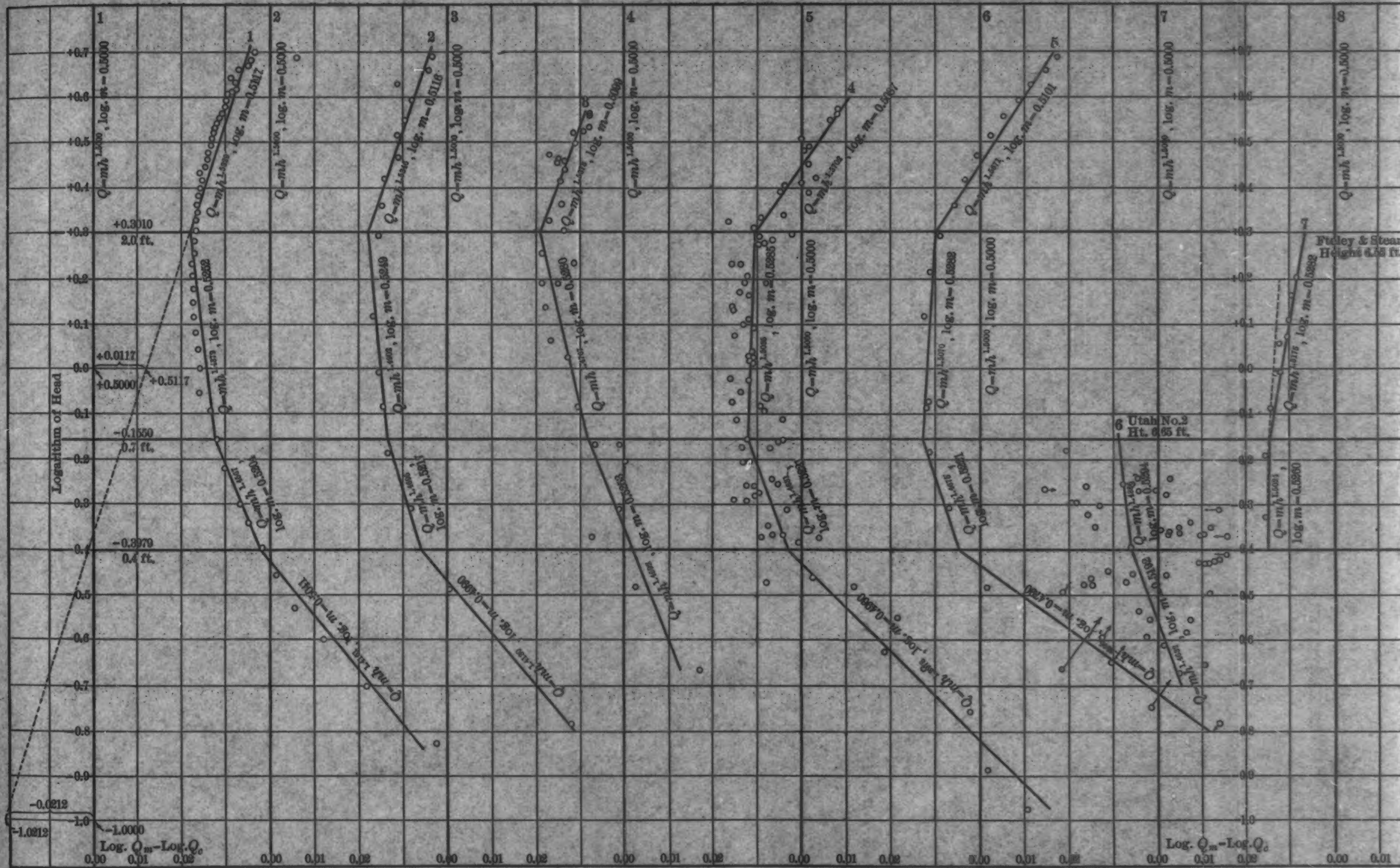
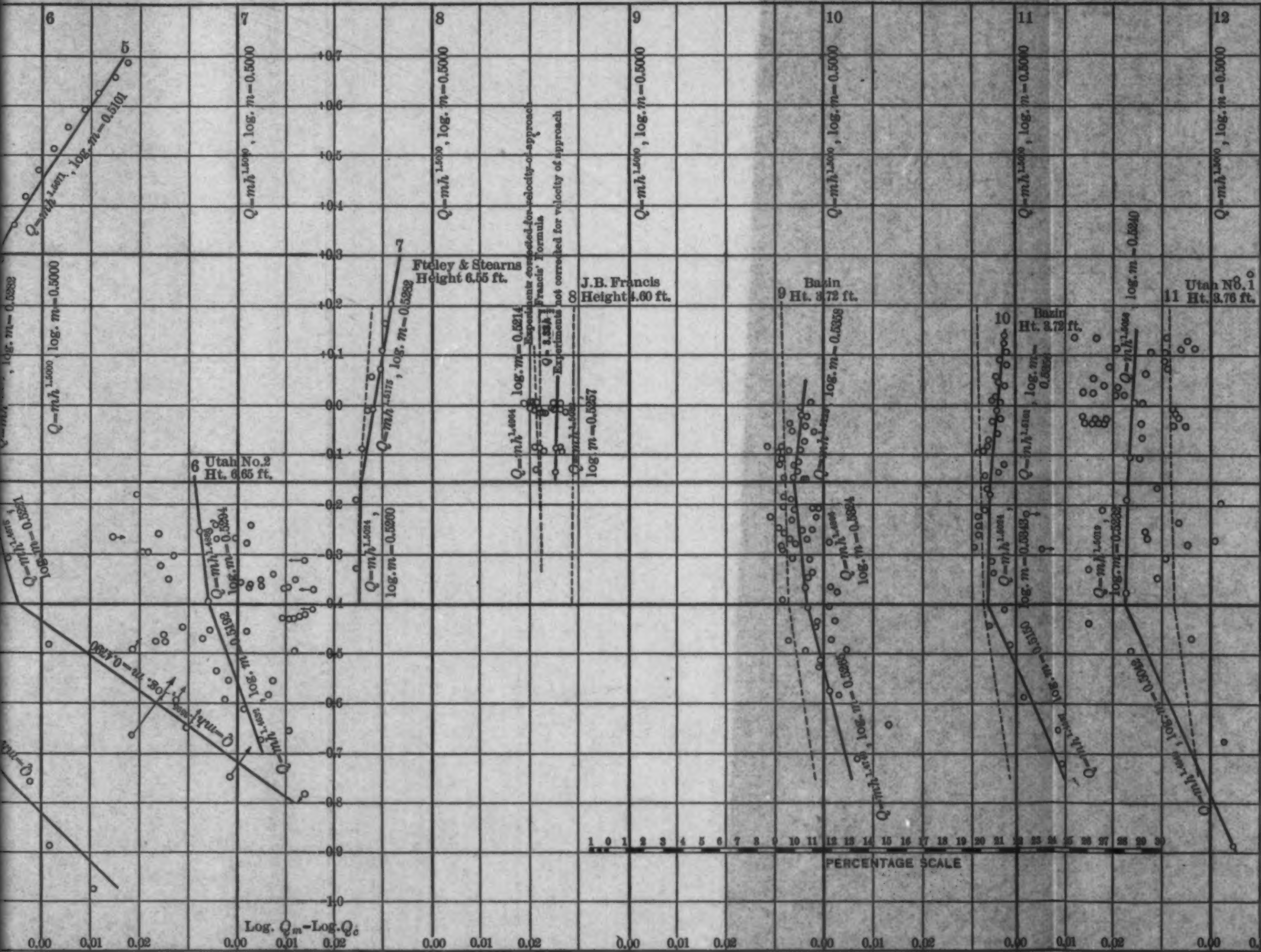


FIG. 12.—WEIR NO. 2, DIVERTING DEVICE, AND WASTE-WAY OF HYDRAULIC LABORATORY OF UNIVERSITY OF UTAH.



Logarithm of Head
+0.3010
2.0 ft.
+0.0117
+0.5000
+0.5117
-0.1550
0.7 ft.
-0.3979
0.4 ft.
-0.0212
-1.0212
-1.0000
Log. Q_m - Log. Q_c
Utah No. 2
Ht. 0.65 ft.
Fteley & Stearns
Height 0.55 ft.
$Q = mh^{1.5000}$, log. $m = 0.5000$



Ftéley & Stearns
Height 6.55 ft.
J.B. Francis
Height 4.60 ft.
Bazin
Ht. 3.72 ft.
Utah No.2
Ht. 6.65 ft.
Utah No.1
Ht. 3.76 ft.
Experiments corrected for velocity of approach
Francis' Formula
Experiments not corrected for velocity of approach
PERCENTAGE SCALE
Log. Q_m−Log. Q_c

PLATE XXVIIc.
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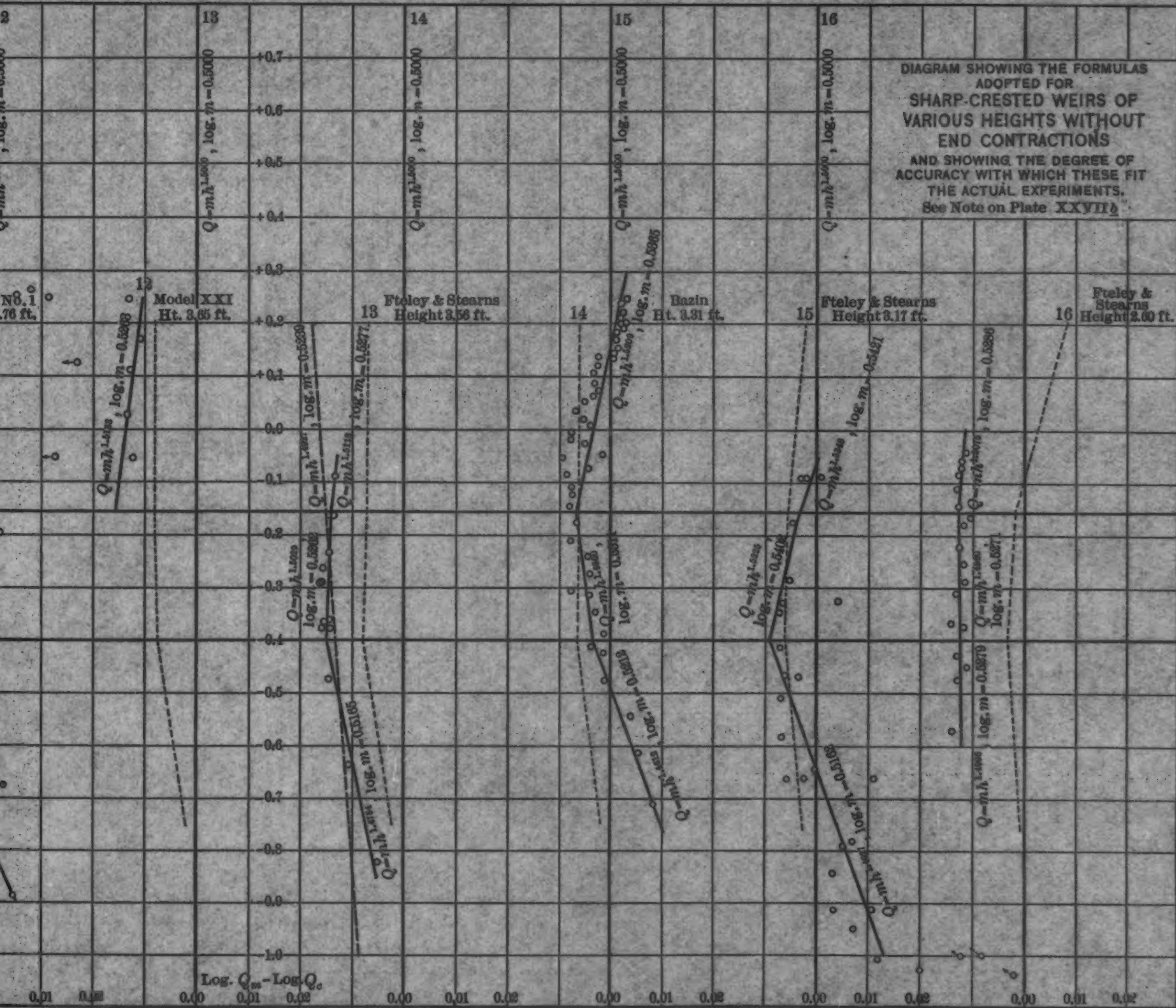
LYMAN ON
WEIR MEASUREMENT OF
STREAM FLOW.

DIAGRAM SHOWING THE FORMULAS ADOPTED FOR
SHARP-CRESTED WEIRS OF VARIOUS HEIGHTS WITHOUT END CONTRACTIONS
AND SHOWING THE DEGREE OF ACCURACY WITH WHICH THESE FIT THE ACTUAL EXPERIMENTS.
See Note on Plate XXVIIb.



it, to right and to left, the computed differences of the logarithms of the quantities are plotted.

Example.—On Plate XXVII*a*, Fig. 1, the vertical line having the equation, log. $Q = 1.5000$ log. $h + 0.5000$, or $Q = m\ h^{1.5000}$ (log. $m = 0.5000$), is marked 1. The points plotted with reference to this line are shown in the curve marked 1. The quantities plotted are those in Column 6 of Table 1, and the logarithms of the heads as in Column 3. The quantities in Column 6 are the differences of the logarithms of Q_m (measured) and Q_c (computed by the trial formula).

The horizontal scale of this drawing (as originally plotted) is 0.01 (logarithm) to the inch; these differences, therefore, can be and are plotted to the fourth decimal place. If a drawing similar to that shown by Fig. A, Plate XXVII*b*, were made to this same scale, the line, *ab*, between the lines, log. $h = -1.0$ and log. $h = 0.71$, that is, between the lines representing heads of 0.1 ft. and 5.1 ft. (values well within the limits of Plates XXVII*a* and XXVII*b*), would be 22 ft. long. To show, on such a scale, all the matter on Plates XXVII*a* and XXVII*b* would require a drawing 33 ft. long if the curves were placed at the same horizontal distance from each other, though the length of Plate XXVII*, as originally drawn, is only 11 ft. On the two parts of this plate the lines and curves are located so that they may be used and studied conveniently; but, if one curve occupied a horizontal distance of 22 ft., it would be impossible to get results of value from its points, as, in most cases, they would be many feet apart. Such lines, being practically horizontal, would give very poor points of intersection with the horizontal lines which represent the heads; whereas those used on Plates XXVII*a* and XXVII*b* give good points of intersection.

After plotting these points, right lines were drawn to fit them, and the equations of the lines thus drawn have been determined from their points of intersection with the lines, log. $h = 0$ and log. $h = -1$. As explained previously, the reference line intersects the line, log. $h = -1.0$, at the point, -1.0000, and the line, log. $h = 0.0$, at the point, $+0.5000$.

In Fig. 1, Plate XXVII*a*, it will be observed that the portion of the curve above log. $h = 0.3010$, when produced downward, as shown by the dotted line, intersects the line, log. $h = 0.0000$, at the

* For convenience, Plate XXVII has been divided into two parts, *a* and *b*.

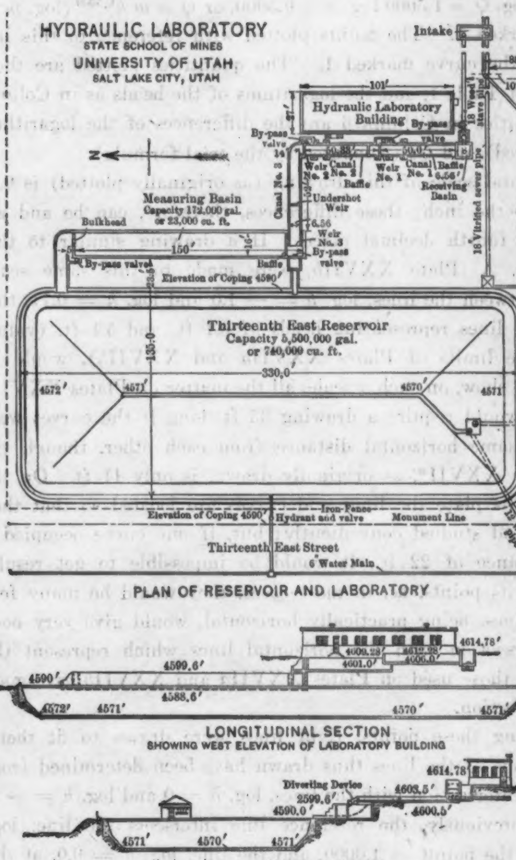
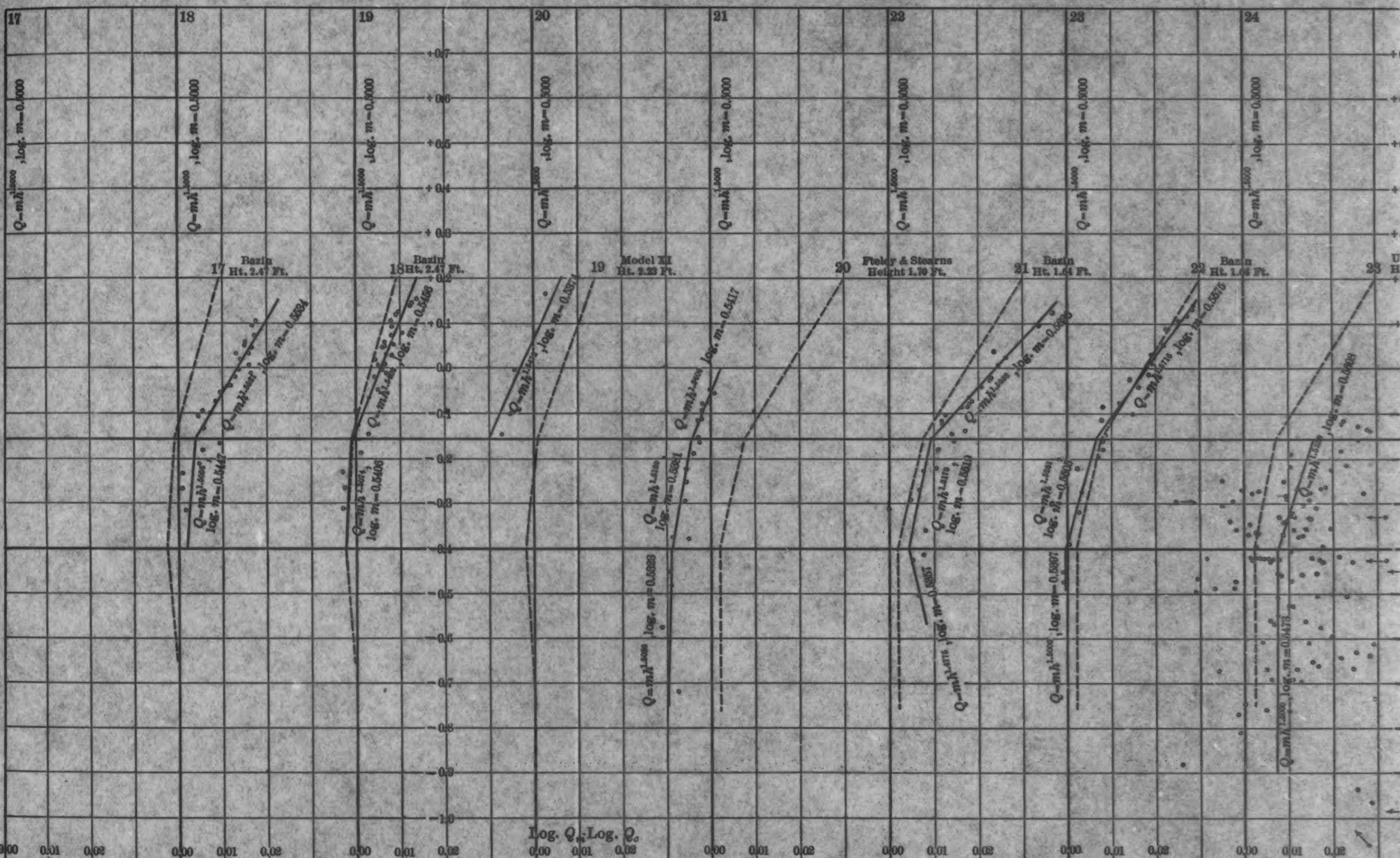
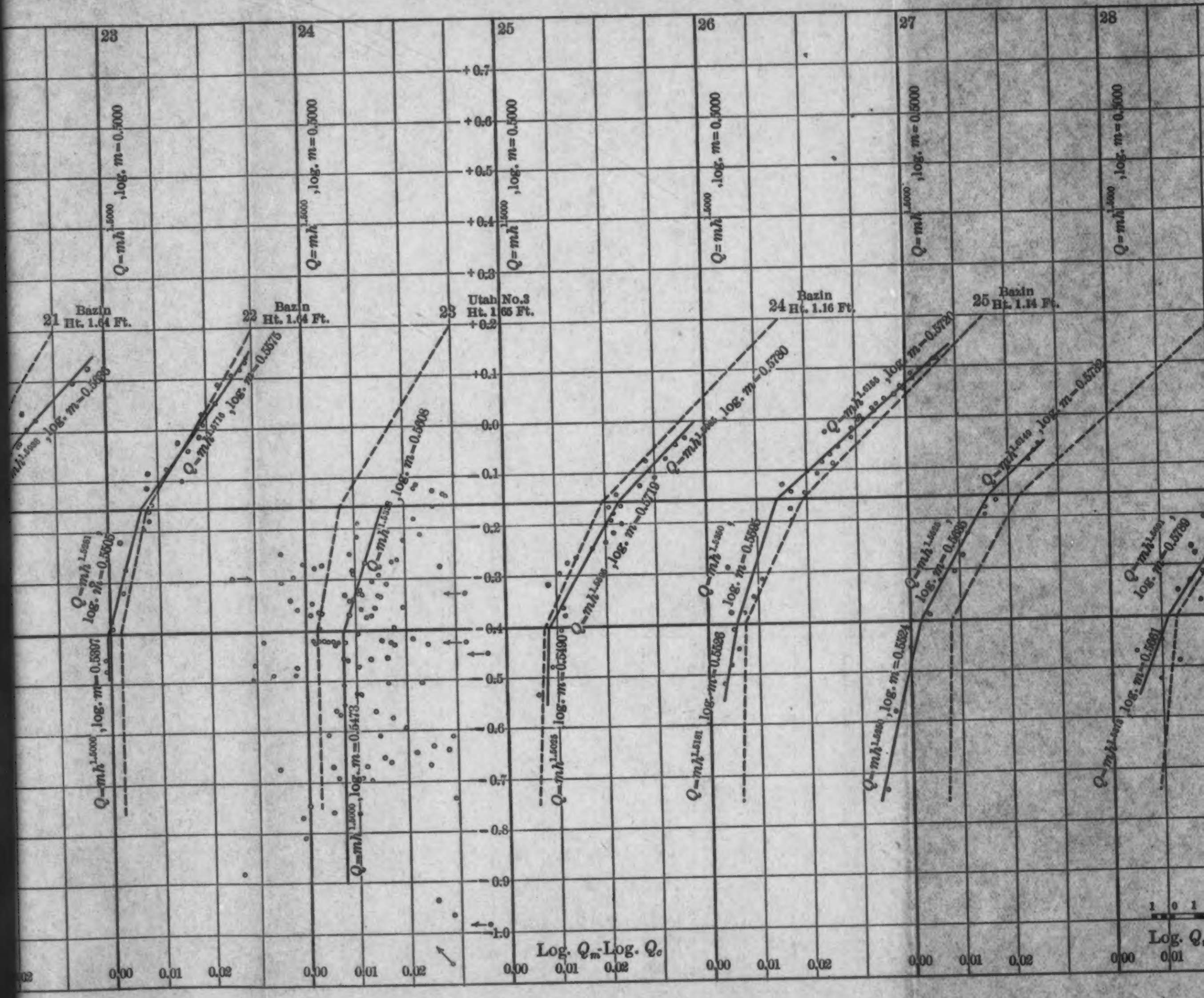
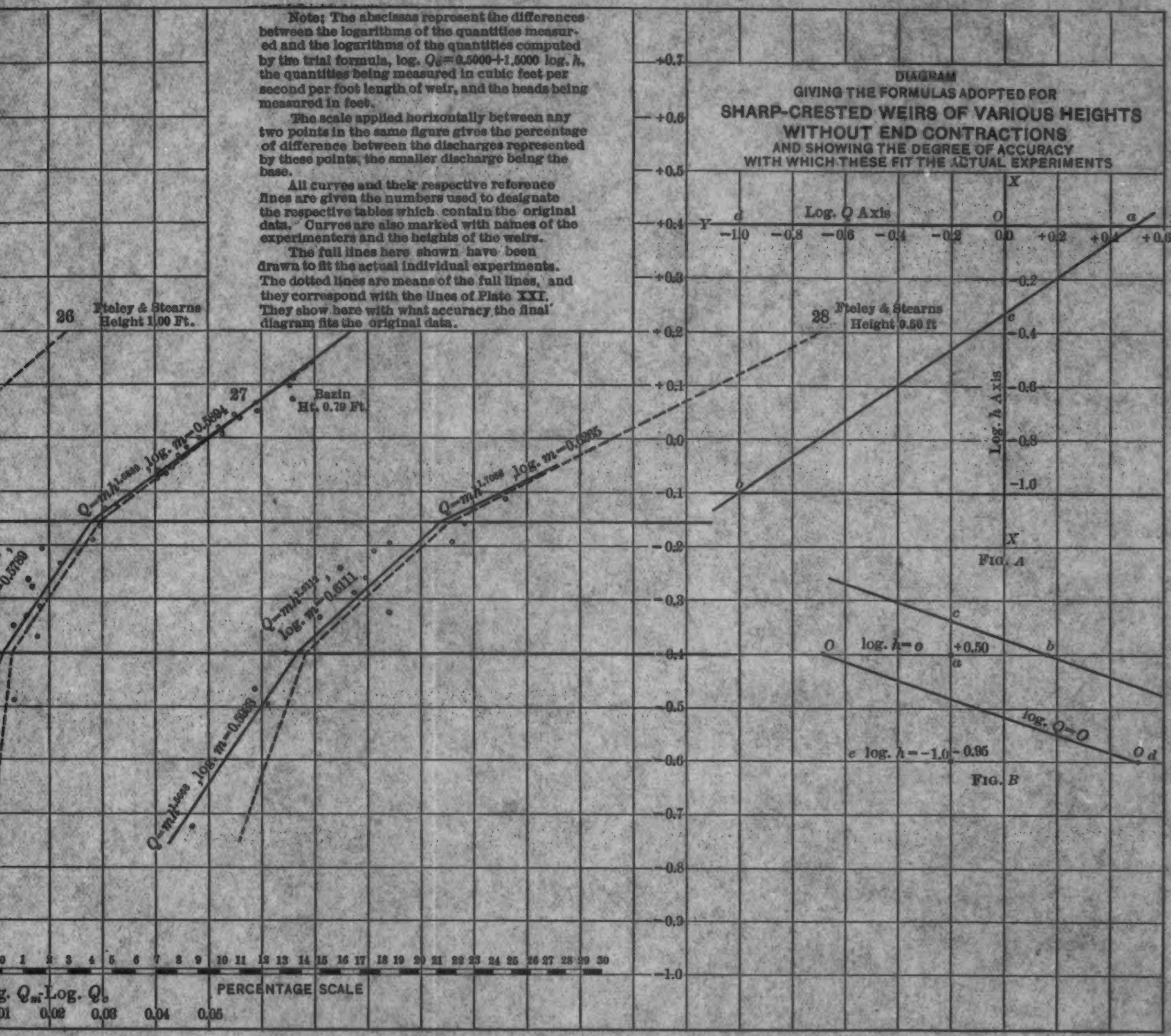

HYDRAULIC LABORATORY
STATE SCHOOL OF MINES
UNIVERSITY OF UTAH
SALT LAKE CITY, UTAH
Intake
88.1′
84.0′
96.0′
107 + 50.0′
101′
Hydraulic Laboratory
Building
By-pass valve
By-pass valve
Weir No. 2
Canal No. 2
Baffle
Weir No. 1
Canal No. 1
Baffle
Receiving Basin
Undershot Weir
Canal No. 3
Weir No. 3
By-pass valve
N
Measuring Basin
Capacity 172,000 gal.
or 23,000 cu. ft.
Bulkhead
150′
By-pass valve
Baffle
Elevation of Coping 4590′
106 + 98.4
105 + 93.9
18 Vitrified sewer pipe
Iron Fence
12 Water Main
8 Water Main
First South Street
Monument Line
Thirteenth East Reservoir
Capacity 5,500,000 gal.
or 740,000 cu. ft.
135.0
330.0
4572′
4571′
4570′
4571
24 C.I. Pipe
104 + 16.8
Gate-house
103 + 59.8
Manhole
Iron Fence
Elevation of Coping 4590′
Hydrant and valve
Monument Line
Vitrified sewer pipe
Monument
Thirteenth East Street
6 Water Main
PLAN OF RESERVOIR AND LABORATORY
4614.78′
4609.28′
4612.28′
4601.0′
4606.0′
4599.6′
4590′
Gate-house
4588.6′
4572′
4571′
4570′
4571′
LONGITUDINAL SECTION
SHOWING WEST ELEVATION OF LABORATORY BUILDING
4614.78
Diverting Device
4603.5′
2599.6′
4590.0′
4600.5′
4571′
4570′
4571′
CROSS-SECTION
SHOWING SOUTH ELEVATION OF LABORATORY BUILDING

FIG. 13.


17 Bazin Ht. 2.47 Ft.
18 Bazin Ht. 2.47 Ft.
19 Model XI Ht. 2.23 Ft.
20 Fteley & Stearns Height 1.70 Ft.
21 Bazin Ht. 1.64 Ft.
22 Bazin Ht. 1.64 Ft.
Log. Q - Log. Q



23
24
25
26
27
28
21 Bazin Ht. 1.64 Ft.
22 Bazin Ht. 1.64 Ft.
23 Utah No.3 Ht. 1.65 Ft.
24 Bazin Ht. 1.16 Ft.
25 Bazin Ht. 1.14 Ft.
Q=mh^1.5000, log. m=0.5000
+0.7
+0.6
+0.5
+0.4
+0.3
+0.2
+0.1
0.0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
-0.7
-0.8
-0.9
-1.0
Log. Q_m-Log. Q_c
0.00
0.01
0.02


Note: The abscissas represent the differences between the logarithms of the quantities measured and the logarithms of the quantities computed by the trial formula, log. Q_c = 0.5000+1.5000 log. h, the quantities being measured in cubic feet per second per foot length of weir, and the heads being measured in feet.
The scale applied horizontally between any two points in the same figure gives the percentage of difference between the discharges represented by these points, the smaller discharge being the base.
All curves and their respective reference lines are given the numbers used to designate the respective tables which contain the original data. Curves are also marked with names of the experimenters and the heights of the weirs.
The full lines here shown have been drawn to fit the actual individual experiments. The dotted lines are means of the full lines, and they correspond with the lines of Plate XXI. They show here with what accuracy the final diagram fits the original data.
DIAGRAM
GIVING THE FORMULAS ADOPTED FOR
SHARP-CRESTED WEIRS OF VARIOUS HEIGHTS
WITHOUT END CONTRACTIONS
AND SHOWING THE DEGREE OF ACCURACY
WITH WHICH THESE FIT THE ACTUAL EXPERIMENTS
26
Fteley & Stearns
Height 1.00 Ft.
27
Bazin
Ht. 0.79 Ft.
28
Fteley & Stearns
Height 0.50 ft
$Q=mh^{1.6$
log. m=0.5894
$Q=mh^{1.7088}$
log. m=0.6265
0.5769
$Q=mh^{1.8$
log. m=0.6111
$Q=mh^{1.5068}$
log. m=0.5958
Log. Q Axis
Log. h Axis
FIG. A
log. h=o
+0.50
log. Q=O
e log. h=−1.0
−0.95
FIG. B
PERCENTAGE SCALE
Log. Q_m-Log. Q_c

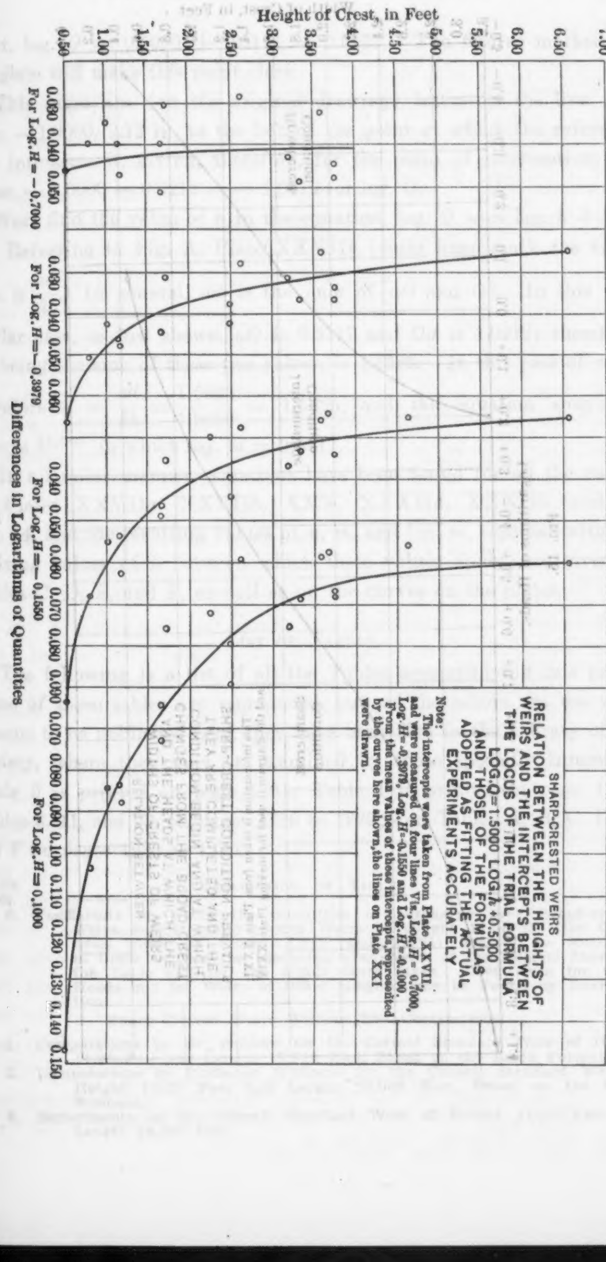

Height of Crest, in Feet
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
5.00
5.50
6.00
6.50
7.00
SHARP-CRESTED WEIRS
RELATION BETWEEN THE HEIGHTS OF WEIRS AND THE INTERCEPTS BETWEEN THE LOCUS OF THE TRIAL FORMULA
LOG.Q=1.5000 LOG.h+0.5000
AND THOSE OF THE FORMULAS ADOPTED AS FITTING THE ACTUAL EXPERIMENTS ACCURATELY
Note:
The intercepts were taken from Plate XXVII, and were measured on four lines Viz., Log.H= 0.7000, Log.H=-0.3979, Log.H=-0.1550 and Log.H=-0.1000.
From the mean values of these intercepts, represented by the curves here shown, the lines on Plate XXI were drawn.
0.030 0.040 0.050
For Log.H=-0.7000
0.030 0.040 0.050 0.060
For Log.H=-0.3979
0.040 0.050 0.060 0.070 0.080 0.090
For Log.H=-0.1550
0.070 0.080 0.090 0.100 0.110 0.120 0.130 0.140 0.150
For Log.H=0.1000
Differences in Logarithms of Quantities

FIG. 14.

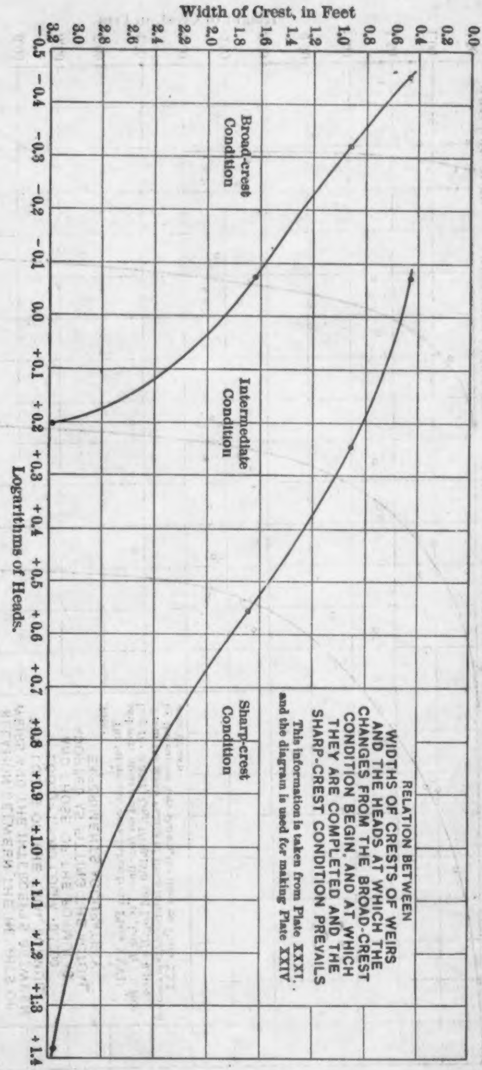

Width of Crest, in Feet
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
−0.5
−0.4
−0.3
−0.2
−0.1
0.0
+0.1
+0.2
+0.3
+0.4
+0.5
+0.6
+0.7
+0.8
+0.9
+1.0
+1.1
+1.2
+1.3
+1.4
Logarithms of Heads.
Broad-crest Condition
Intermediate Condition
Sharp-crest Condition
RELATION BETWEEN WIDTHS OF CRESTS OF WEIRS AND THE HEADS AT WHICH THE CHANGES FROM THE BROAD-CREST CONDITION BEGIN, AND AT WHICH THEY ARE COMPLETED AND THE SHARP-CREST CONDITION PREVAILS
This information is taken from Plate XXXI, and the diagram is used for making Plate XXIV

Fig. 15.

point, log. $Q = 0.5000 + 0.0117 = 0.5117$. The figures marked on the plate will make this point clear.

This same line (on the original drawing) intersects the line, log. $h = -1.0000$, 2.12 in. to the left of the point at which the reference line intersects it, giving, therefore, for the point of intersection, the value, $-1.0000 - 0.0212 = -1.0212$ for log. Q.

Next find the value of n in the equation, log. $Q = n$ log. $h +$ log. m. Referring to Fig. A, Plate XXVIIb (right hand end), the value of n is $\frac{ad}{db}$. In general, ad is the sum of aO and Od. In this particular case, as just shown, aO is 0.5117 and Od is 1.0212; therefore, ad, being the sum of these two values, is 1.5329. In this case $db = 1$, therefore $n = \frac{ad}{db} = \frac{1.5329}{1.0000} = 1.5329$, and the equation sought is $Q = m\,h^{1.5329}$, in which log. $m = 0.5117$.

In a similar manner, equations have been found for all the curves on Plates XXVIIa, XXVIIb, XXX, XXXIIa, XXXIIb, and on Fig. 14, and the resulting values of n, m, and log. m, together with the limiting values of h between which these values apply, are given in Tables C, D, E, and F, as well as on the curves on the plates.

List of Tables.

The following is a list of all the Tables accompanying this paper. Some of these tables are reproduced, but, as the others are too voluminous to be published here, they have been filed in the Library of the Society, where they may be examined by those specially interested. Table 0 is printed on page 1201; Table 1 is printed on page 1225; Tables 6, 11, and 23, on pages 1226 to 1229; and Tables A, B, C, D, E, and F on pages 1230 to 1240.

Table No.	Description of Tables.
0.	Coefficients by which the Quantities of Discharge over Broad-crested Weirs and Irregular-crested Weirs of Height 11.25 ft. (the Quantities given on Plates XXIV and XXVI) are to be Multiplied in Order to Give the Discharge over Weirs of the Heights Shown in the Table and for the Heads there given. Coefficients for Other Heads and for Weirs of Other Heights may be Found by Interpolation.
	Sharp-Crested Weirs Without End Contractions.
1.	Computations by Dr. Schoder for the Cornell Standard Weir of Height 11.25 Feet and Length 15.995 Feet, Based on the Bazin Formula.
2.	Computations by Professor Williams for the Cornell Standard Weir of Height 11.25 Feet and Length 16.005 Feet, Based on the Bazin Formula.
3.	Experiments on the Cornell Standard Weir of Height 11.25 Feet and Length 16.005 Feet.

Broad-Crested Weirs Without End Contractions.

(Experiments in the Hydraulic Laboratory of Cornell University. Models Built Over the Upper Standard Weir, Height 11.25 Feet, Length 16.00 Feet.)

Table No.	Description of Tables.—(*Continued.*)
40.	Cornell Model XLI. Width 19¾ In., or 1.65 Ft.
41.	Cornell Model XLVI. Width 3.17 Ft.
42.	Cornell Model XLV. Width 5.85 Ft.
43.	Cornell Model XLIV. Width 8.96 Ft.
44.	Cornell Model XLII. Width 12.25 Ft.
45.	Cornell Model XLIII. Width 16.29 Ft.
46.	Cornell Model XLIIIa. Width 16.29 Ft. (Model XLIII planed.)
	Irregular-Shaped Weirs and Dams.
	(Experiments in the Hydraulic Laboratory of Cornell University.)
47.	Cornell Model XI.
48.	Cornell Model XXII. Cambria Weir.
49.	Cornell Model XXVI. Cambria Weir.
50.	Cornell Model XXIII. Cambria Weir.
51.	Cornell Model XXIV. Cambria Weir.
52.	Cornell Model XXV. Cambria Weir.
53.	Cornell Model XXVII. Lawrence Dam.
54.	Cornell Model XX.
55.	Cornell Model XXVII. Lawrence Dam.
56.	Mr. Francis' Model. Lawrence Dam.
57.	Cornell Model XXX. Plattsburg Dam.
58.	Cornell Model XXXIII. Plattsburg Dam.
59.	Cornell Model XXXIV. Plattsburg Dam.
60.	Cornell Model XXXV. Chambly Dam.
61.	Cornell Model XXXVI. Chambly Dam.
62.	Cornell Model XXXVIII. Dolgeville Dam.
63.	Cornell Model XXXIX. Dolgeville Dam.
64.	Cornell Model XLVIII.
65.	Cornell Model L.
66.	Cornell Model LI.
67.	Cornell Model XLIX.
A.	Heads on the Standard Weir (Height 11.25 Feet) as Measured by Different Methods Simultaneously.
	Heads on the Standard Weir (Height 11.25 Feet) as Measured by the Standard-Tube Piezometer and the 10-Ft. Tape Simultaneously.
	Heads on the Standard Weir (Height 6.65 Feet) as Measured by Different Methods Simultaneously.
B.	Heads Measured Simultaneously With the Hook-Gauge in the Bazin Pit and With the 15-Ft. Tape.
C.	Values of n, m, and log. m, With the Limits of Head Between Which These Apply, to Be Used in the Equation, $Q = m\ h^n$, for Sharp-Crested Weirs of Various Heights.
D.	Values of n, m, and log. m, With the Limits of Head Between Which These Apply, to Be Used in the Equation, $Q = m\ h^n$, for Broad-Crested Weirs of Various Widths.
E.	Values of n, m, and log. m, With the Limits of Head Between Which These Apply, to Be Used in the Equation, $Q = m\ h^n$, for Irregular-Crested Weirs With Sections of Right Lines.
F.	Values of n, m, and log. m, With the Limits of Head Between Which These Apply, to Be Used in the Equation, $Q = m\ h^n$, for Irregular-Crested Weirs With Sections of Right Lines and Curves.

F.—Actual Determination of the Formulas.

Formulas of the proposed form, namely, $Q = m\ h^n$, have been prepared for four different classes of weirs without end contractions:

(1) Sharp-crested weirs;

(2) Broad-crested weirs;

(3) Irregular weirs with cross-sections of right lines;

(4) Irregular weirs with cross-sections of right lines and curves.

The values of the constants for the formulas derived are given in Tables C, D, E, and F.

As measuring the head on the crest of a weir with a "tape" is perhaps the simplest and best method in actual field practice, all heads

measured otherwise than with a tape, except where specifically noted, have been reduced to equivalent tape readings by using the curves on Plates XXXIII and XXXIV. The distances up stream from the crest at which these tape readings have been taken are given, with the tabulation of the data, in Tables 1 to 28 and 38 to 67, inclusive, and also on the discharge diagrams, Plates XXI, XXIV, XXV, and XXVI.

(1).—For Sharp-Crested Weirs.

(*a*).—*General Statement.*—Equations are herein derived and diagrams devised for determining the discharge over all the weirs of this class on which accurate experiments, on a large scale, have been made. The adopted equations for these weirs, and the limits of the heads between which each may be applied, are given in Table C.

The experiments in which the water was measured volumetrically, or with weirs rated by volumetric measurement, include those of Francis, Fteley and Stearns, Bazin, and those made in the hydraulic laboratory of the University of Utah. These experiments have been made with weirs having heights ranging from the Fteley and Stearns weir, 0.50 ft. high (5 ft. long), to the Utah weir No. 2, 6.65 ft. high (6.55 ft. long). The experiments of Francis were made on only one weir of this class, and it was 4.60 ft. high and 9.992 ft. long.*

The Cornell 6-ft. Francis piezometer was constructed for the purpose of duplicating the apparatus used by Francis for measuring heads. Observe that the curve on Plate XXXIV, as determined by the readings of this piezometer, runs across nearly all the other curves for heads of less than 1 ft. It seems hardly reasonable that the readings for these small heads should increase so much more rapidly, as the zero point is approached, than do those of the other devices; and as the pressure orifice for this piezometer was between the two gates in the lower end of the canal, it is probable that a small leak induced a slight velocity toward the opening. The impact due to this velocity would account for the small increase in the reading. It is for this reason that the heads measured by Mr. Francis and those measured by Messrs. Fteley and Stearns, using this same method, have not been reduced to equivalent tape readings.

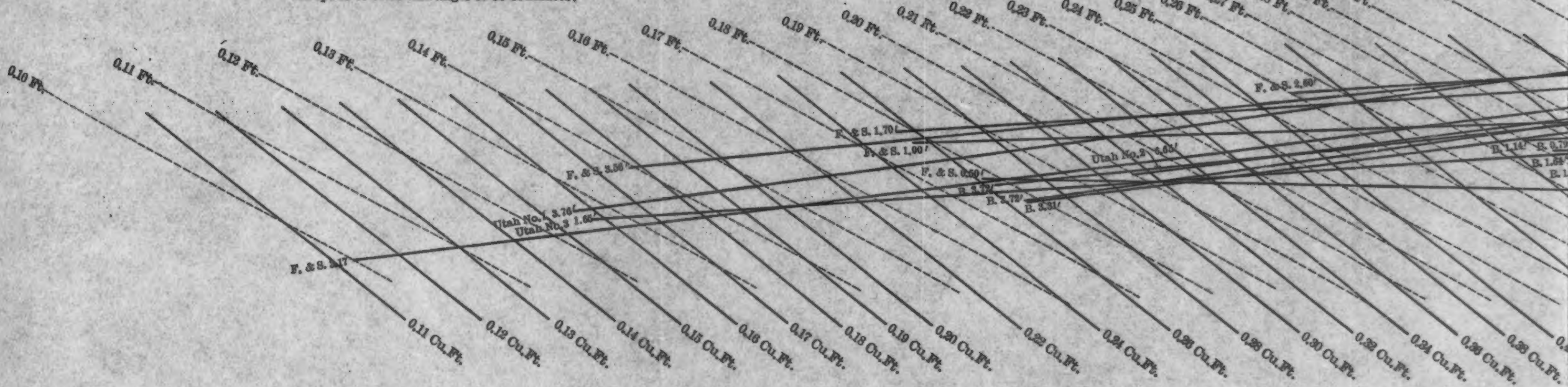
(*b*).—*Francis Experiments.*—The Francis experiments (Table 8) are plotted in Fig. 8, Plate XXVII*a*. The height of this weir was

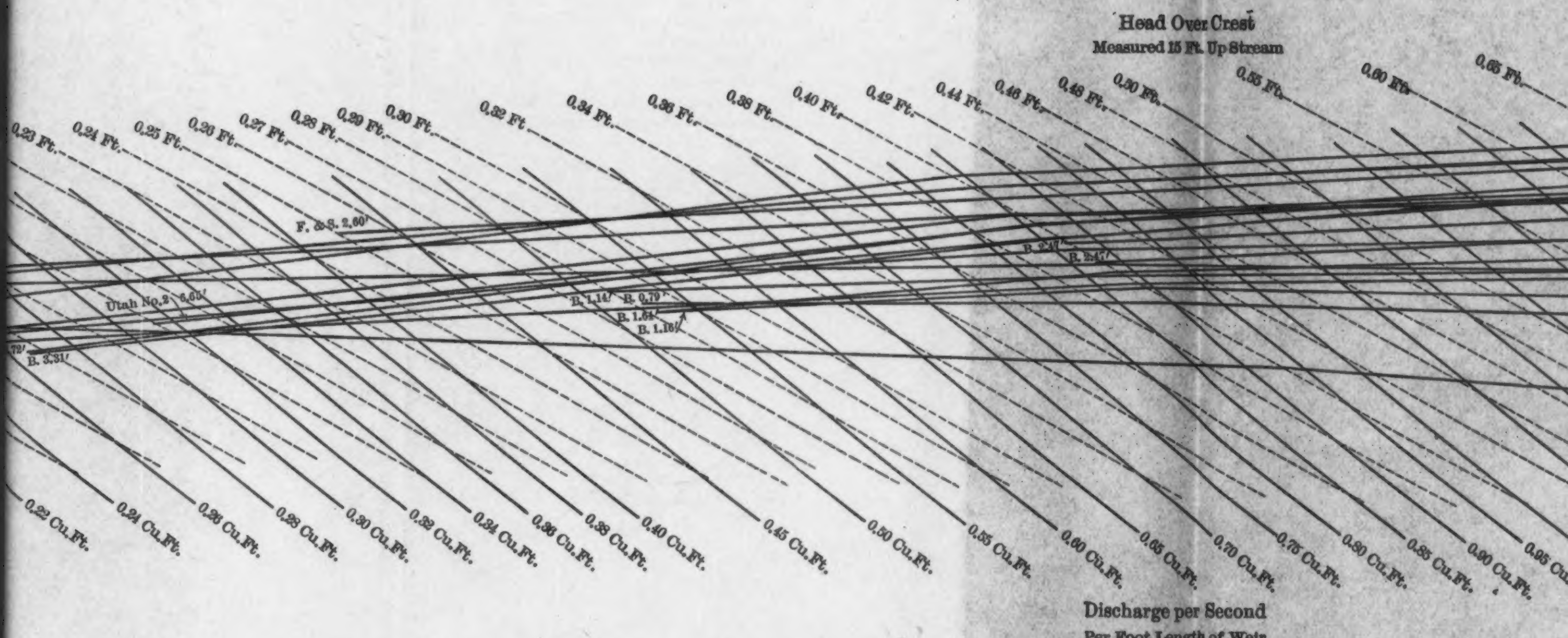
* "Lowell Hydraulic Experiments," pp. 124, 125.

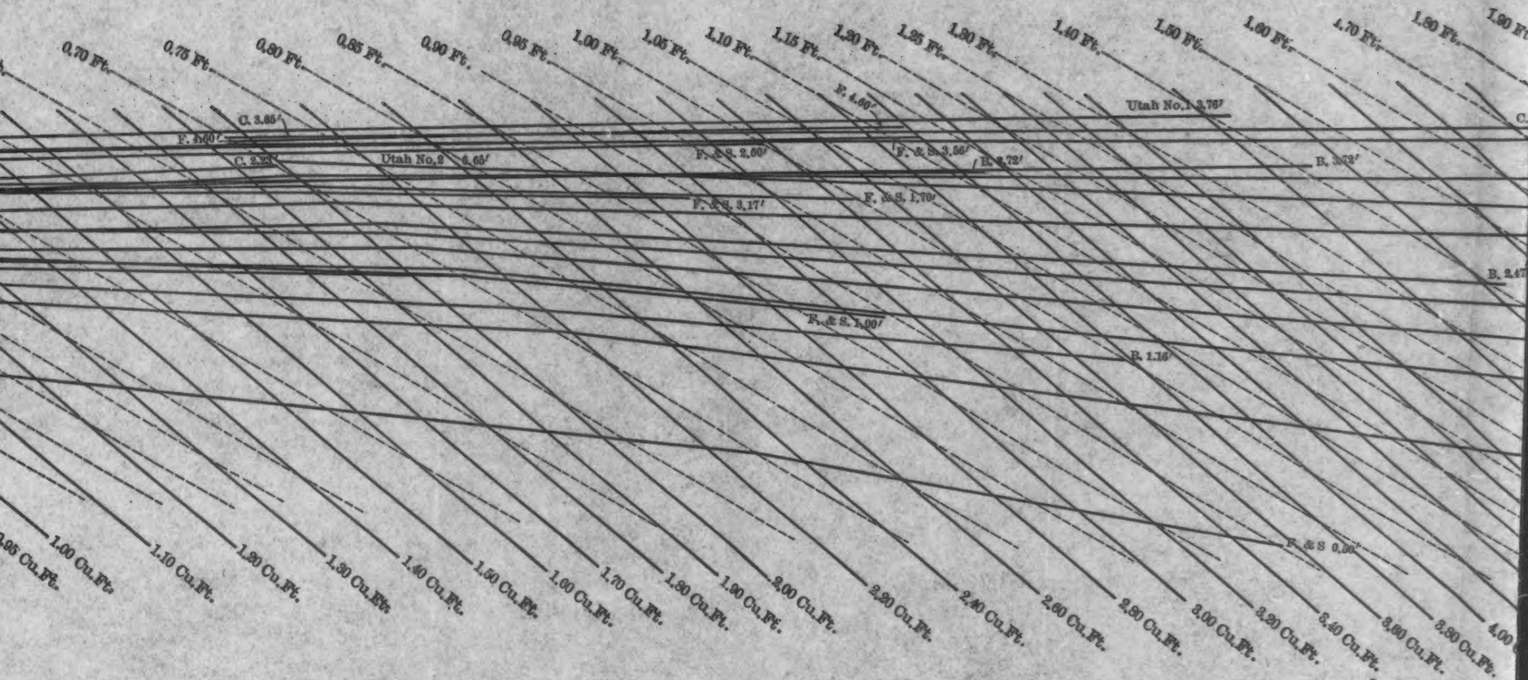
HEADS AND CORRESPONDING DISCHARGES OVER.

SPECIFIC SHARP-CRESTED WEIRS OF VARIOUS HEIGHTS
WITHOUT END CONTRACTIONS.

Note: These curves are plotted from the experiments made by Bazin, Fteley and Stearns, Francis, and in the Hydraulic Laboratories of Cornell University and the University of Utah. Mean curves for practical use, derived from the same data, are plotted on Plate XXI. This plate is identical with Plate XXIX, except as to scale and angle of co-ordinates.




Head Over Crest
Measured 15 Ft. Up Stream
0,23 Ft.
0,24 Ft.
0,25 Ft.
0,26 Ft.
0,27 Ft.
0,28 Ft.
0,29 Ft.
0,30 Ft.
0,32 Ft
0,34 Ft.
0,36 Ft.
0,38 Ft.
0,40 Ft.
0,42 Ft.
0,44 Ft.
0,46 Ft.
0,48 Ft.
0,50 Ft.
0,55 Ft.
0,60 Ft.
0,65 Ft.
F. & S. 2,60'
Utah No.2
B. 3.31'
B. 1.14'
B. 0.79'
B. 1.54'
B. 1.16'
B. 2.47'
B. 2.47'
0,22 Cu.Ft.
0,24 Cu.Ft.
0,26 Cu.Ft.
0,28 Cu.Ft.
0,30 Cu.Ft.
0,32 Cu.Ft.
0,34 Cu.Ft.
0,36 Cu.Ft.
0,38 Cu.Ft.
0,40 Cu.Ft.
0,45 Cu.Ft.
0,50 Cu.Ft.
0,55 Cu.Ft.
0,60 Cu.Ft.
0,65 Cu.Ft.
0,70 Cu.Ft.
0,75 Cu.Ft.
0,80 Cu.Ft.
0,85 Cu.Ft.
0,90 Cu.Ft.
Discharge per Second
Per Foot Length of Weir



0.70 Ft.
0.75 Ft.
0.80 Ft.
0.85 Ft.
0.90 Ft.
0.95 Ft.
1.00 Ft.
1.05 Ft.
1.10 Ft.
1.15 Ft.
1.20 Ft.
1.25 Ft.
1.30 Ft.
1.40 Ft.
1.50 Ft.
1.60 Ft.
1.70 Ft.
1.80 Ft.
1.90 Ft.
Utah No.1 3.76′
C. 3.65′
F. 4.60′
C. 2.23′
Utah No.2 6.65′
F. 4.60′
F. & S. 2.60′
F. & S. 3.56′
B. 3.72′
B. 3.72′
F. & S. 3.17′
F. & S. 1.70′
B. 2.47
F. & S. 1.00′
B. 1.16′
F. & S 0.50′
1.00 Cu.Ft.
1.10 Cu.Ft.
1.20 Cu.Ft.
1.30 Cu.Ft.
1.40 Cu.Ft.
1.50 Cu.Ft.
1.60 Cu.Ft.
1.70 Cu.Ft.
1.80 Cu.Ft.
1.90 Cu.Ft.
2.00 Cu.Ft.
2.20 Cu.Ft.
2.40 Cu.Ft.
2.60 Cu.Ft.
2.80 Cu.Ft.
3.00 Cu.Ft.
3.20 Cu.Ft.
3.40 Cu.Ft.
3.60 Cu.Ft.
3.80 Cu.Ft.

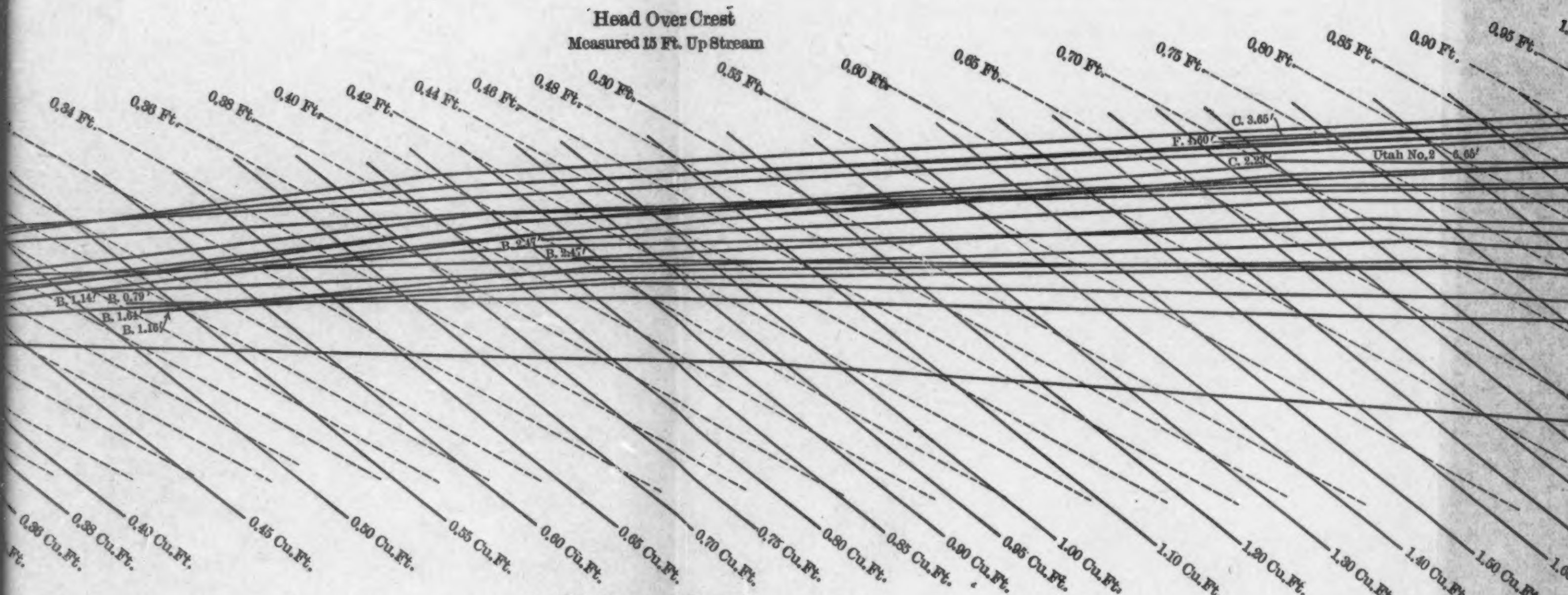

Head Over Crest
Measured 15 Ft. Up Stream
0.34 Ft.
0.36 Ft.
0.38 Ft.
0.40 Ft.
0.42 Ft.
0.44 Ft.
0.46 Ft.
0.48 Ft.
0.50 Ft.
0.55 Ft.
0.60 Ft.
0.65 Ft.
0.70 Ft.
0.75 Ft.
0.80 Ft.
0.85 Ft.
0.90 Ft.
0.95 Ft.
C. 3.65'
C. 2.23'
Utah No. 2
B. 2.47'
B. 2.47'
B. 1.14'
B. 0.79'
B. 1.64'
B. 1.16'
0.36 Cu.Ft.
0.38 Cu.Ft.
0.40 Cu.Ft.
0.45 Cu.Ft.
0.50 Cu.Ft.
0.55 Cu.Ft.
0.60 Cu.Ft.
0.65 Cu.Ft.
0.70 Cu.Ft.
0.75 Cu.Ft.
0.80 Cu.Ft.
0.85 Cu.Ft.
0.90 Cu.Ft.
0.95 Cu.Ft.
1.00 Cu.Ft.
1.10 Cu.Ft.
1.20 Cu.Ft.
1.30 Cu.Ft.
1.40 Cu.Ft.
1.50 Cu.Ft.
Discharge per Second
Per Foot Length of Weir

PLATE XXVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVII, No. 1304.
LYMAN ON
WEIR MEASUREMENT OF
STREAM FLOW.

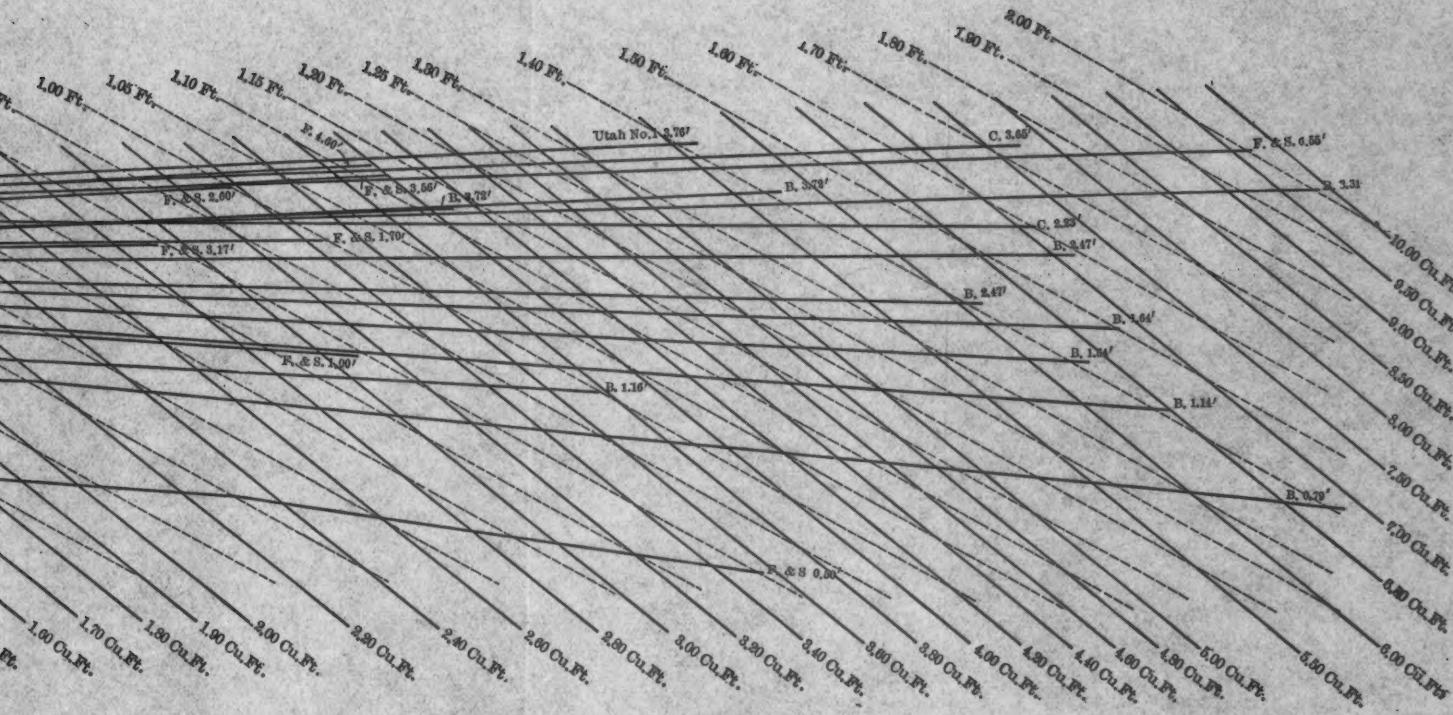


TABLE 1.*—SHARP-CRESTED WEIRS.

Computations by Dr. Schoder, Based on the Bazin Formula, for a Weir of Height 11.25 ft.

(1) No. of exp.	(2) Assumed head.	(3) 15-ft. tape head.	(4) Log. Q_c	(5) Log. Q_B	(6) Log. Q_B —Log. Q_c	(7) Q_c	(8) Q_B
	5.00		1.5485	1.5846	0.0361	35.361	38.421
	4.90		1.5353	1.5709	0.0356	34.300	37.234
	4.80		1.5217	1.5569	0.0352	33.243	36.049
	4.70		1.5080	1.5426	0.0346	32.111	34.881
	4.60		1.4940	1.5282	0.0342	31.189	33.742
	4.50		1.4798	1.5132	0.0334	30.186	32.600
	4.40		1.4651	1.4980	0.0329	29.181	31.479
	4.30		1.4501	1.4825	0.0324	28.190	30.377
	4.20		1.4348	1.4667	0.0321	27.215	29.286
	4.10		1.4191	1.4504	0.0313	26.248	28.213
	4.00		1.4030	1.4338	0.0308	25.293	27.155
	3.90		1.3865	1.4168	0.0303	24.350	26.111
	3.80		1.3696	1.3994	0.0298	23.421	25.085
	3.70		1.3523	1.3815	0.0292	22.506	24.072
	3.60		1.3344	1.3631	0.0287	21.597	23.074
	3.50		1.3160	1.3443	0.0283	20.701	22.097
	3.40		1.2972	1.3250	0.0278	19.824	21.135
	3.30		1.2778	1.3051	0.0273	18.958	20.187
	3.20		1.2577	1.2845	0.0268	18.101	19.253
	3.10		1.2370	1.2634	0.0264	17.258	18.339
	3.00		1.2157	1.2416	0.0259	16.432	17.441
	2.90		1.1935	1.2191	0.0256	15.613	16.561
	2.80		1.1707	1.1958	0.0251	14.815	15.697
	2.70		1.1471	1.1718	0.0247	14.031	14.815
	2.60		1.1224	1.1468	0.0244	13.256	14.021
	2.50		1.0969	1.1209	0.0240	12.250	13.209
	2.40		1.0703	1.0940	0.0237	11.760	12.417
	2.30		1.0426	1.0659	0.0233	11.031	11.638
	2.20		1.0136	1.0367	0.0231	10.328	10.882
	2.10		0.9833	1.0061	0.0228	9.623	10.142
	2.00		0.9515	0.9741	0.0226	8.943	9.422
	1.90		0.9181	0.9406	0.0225	8.281	8.722
	1.80		0.8828	0.9052	0.0224	7.635	8.039
	1.70		0.8456	0.8678	0.0222	7.008	7.376
	1.60		0.8062	0.8284	0.0222	6.400	6.736
	1.50		0.7640	0.7864	0.0224	5.808	6.115
	1.40		0.7192	0.7415	0.0223	5.238	5.515
	1.30		0.6709	0.6965	0.0224	4.687	4.937
	1.20		0.6186	0.6416	0.0230	4.155	4.381
	1.10		0.5619	0.5854	0.0235	3.646	3.849
	1.00		0.5000	0.5239	0.0239	3.162	3.342
	0.90		0.4313	0.4551	0.0238	2.700	2.852
	0.80		0.3545	0.3806	0.0261	2.262	2.402
	0.70		0.2675	0.2951	0.0276	1.851	1.973
	0.60		0.1671	0.1970	0.0299	1.469	1.574
	0.50		0.0483	0.0814	0.0331	1.118	1.206
	0.45		—0.0202	0.0150	0.0352	0.955	1.085
	0.40		—0.0970	—0.0590	0.0380	0.800	0.873
	0.35		—0.1840	—0.1425	0.0415	0.655	0.720
	0.30		—0.2844	—0.2385	0.0459	0.519	0.577
	0.25		—0.4301	—0.3508	0.0523	0.395	0.446
	0.20		—0.5485	—0.4865	0.0620	0.283	0.326
	0.15		—0.7360	—0.6588	0.0772	0.188	0.219
	0.10		—1.0000	—0.8985	0.1065	0.100	0.128

* Table 1 is here reproduced as a sample of the tables numbered 1 to 8, 8A, and 9 to 67. All these tables, together with Tables A, B, C, D, E, and F, are filed in the Library of the Society, where they may be examined by those specially interested.

4.60 ft. The results, obtained by applying the trial formula to the Francis experiments without correcting for velocity of approach, are shown by the points, at the right of the figure, through which a full

TABLE 6.—SHARP-CRESTED WEIRS.

Experiments on the Utah Weir No. 2; Height, 6.65 ft.; Length, 6.55 ft.

(1) No. of exp.	(2) Head.	(3) 15-ft. tape head.	(4) Log. Q_c	(5) Log. Q_m	(6) Log. Q_m −Log. Q_c	(7) Q_c	(8) Q_m
1		0.656	0.2253	0.2450	0.0197	1.680	1.758
2		0.574	0.1384	0.1813	0.0429	1.375	1.518
3		0.572	0.1361	0.1717	0.0356	1.368	1.485
4		0.560	0.1223	0.1547	0.0324	1.325	1.428
5		0.559	0.1211	0.1553	0.0342	1.322	1.430
6		0.550	0.1106	0.1345	0.0239	1.290	1.363
7		0.543	0.1022	0.1169	0.0147	1.265	1.309
8		0.540	0.0986	0.1383	0.0397	1.255	1.375
9		0.535	0.0926	0.1283	0.0357	1.238	1.344
10		0.528	0.0838	0.1258	0.0420	1.213	1.336
11		0.514	0.0665	0.0224	−0.0441	1.165	1.050
12		0.504	0.0536	0.0748	0.0214	1.131	1.188
13		0.503	0.0524	0.0730	0.0206	1.128	1.183
14		0.500	0.0485	0.0752	0.0267	1.118	1.189
15		0.487	0.0313	0.0853	0.0540	1.075	1.217
16		0.485	0.0286	0.0774	0.0488	1.068	1.195
17		0.476	0.0164	0.0406	0.0242	1.038	1.098
18		0.468	0.0053	−0.0685	−0.0738	1.012	0.854
19		0.463	−0.0016	0.1614	0.1630	0.996	1.450
20		0.457	−0.0101	0.0374	0.0475	0.977	1.090
21		0.446	−0.0260	0.0000	0.0260	0.940	1.000
22		0.445	−0.0274	0.0175	0.0449	0.939	1.041
23		0.443	−0.0304	0.0216	0.0520	0.932	1.051
24		0.439	−0.0363	0.0043	0.0406	0.920	1.010
25		0.435	−0.0422	0.0004	0.0426	0.907	1.001
26		0.433	−0.0453	−0.0004	0.0449	0.901	0.999
27		0.431	−0.0642	0.0022	0.0540	0.895	1.005
28		0.430	−0.0498	−0.0074	0.0424	0.892	0.983
29		0.424	−0.0589	−0.0031	0.0558	0.873	0.993
30		0.416	−0.0714	0.0310	0.1024	0.848	1.074
31		0.409	−0.0824	−0.0343	0.0481	0.827	0.924
32		0.403	−0.0920	−0.0580	0.0340	0.809	0.875
33		0.395	−0.1051	−0.0942	0.0109	0.785	0.805
34		0.390	−0.1134	−0.0580	0.0554	0.770	0.875
35		0.378	−0.1338	−0.0804	0.0534	0.735	0.831
36		0.378	−0.1338	−0.0841	0.0497	0.735	0.824
37		0.376	−0.1372	−0.0846	0.0526	0.729	0.823
38		0.376	−0.1372	−0.0846	0.0526	0.729	0.823
39		0.376	−0.1372	−0.0857	0.0515	0.729	0.821
40		0.376	−0.1372	−0.0862	0.0510	0.729	0.820
41		0.376	−0.1372	−0.0862	0.0510	0.729	0.820
42		0.375	−0.1390	−0.0878	0.0512	0.726	0.817
43		0.358	−0.1692	−0.1403	0.0289	0.677	0.724
44		0.351	−0.1820	−0.1403	0.0417	0.658	0.724
45		0.351	−0.1820	−0.1475	0.0345	0.658	0.712
46		0.343	−0.1970	−0.1720	0.0250	0.635	0.673
47		0.342	−0.1990	−0.0794	0.1196	0.632	0.833
48		0.338	−0.2066	−0.1637	0.0329	0.621	0.686
49		0.333	−0.2164	−0.1931	0.0233	0.608	0.641.
50		0.332	−0.2183	−0.1931	0.0252	0.605	0.641
51		0.323	−0.2362	−0.2175	0.0187	0.580	0.606
52		0.320	−0.2423	−0.1904	0.0519	0.572	0.645
53		0.295	−0.2953	−0.2306	−0.0253	0.507	0.478
54		0.292	−0.3019	−0.2660	0.0359	0.499	0.542
55		0.280	−0.3292	−0.2907	0.0385	0.469	0.512
56		0.278	−0.3340	−0.2860	0.0472	0.463	0.517
57		0.262	−0.3726	−0.3261	0.0465	0.424	0.472
58		0.255	−0.3902	−0.3526	0.0376	0.407	0.444
59		0.255	−0.3902	−0.3625	0.0277	0.407	0.434
60		0.247	−0.4110	−0.3696	0.0414	0.388	0.427
61		0.229	−0.4603	−0.3904	0.0699	0.347	0.407
62		0.226	−0.4689	−0.3830	0.0859	0.340	0.414
63		0.224	−0.4747	−0.4449	0.0298	0.335	0.359
64		0.222	−0.4804	−0.4301	0.0503	0.331	0.371
65		0.218	−0.4922	−0.4737	0.0185	0.322	0.336
66		0.212	−0.5105	−0.4660	0.0445	0.309	0.342
67		0.182	−0.6098	−0.3747	0.2351	0.246	0.422
68		0.180	−0.6170	−0.5784	0.0386	0.242	0.264
69		0.151	−0.7315	−0.6904	0.0411	0.186	0.204
70		0.133	−0.8141	−0.7986	0.0155	0.153	0.159
71		0.044	−1.5347	−1.5376	−0.0029	0.030	0.029

TABLE 11.—Sharp-Crested Weirs.

Experiments on the Utah Weir No. 1; Height, 3.72 ft.; Length, 1.77 ft.

(1) No. of exp.	(2) Head.	(3) 15-ft. tape head.	(4) Log. Q_c	(5) Log. Q_m	(6) Log. Q_m —Log. Q_c	(7) Q_c	(8) Q_m
1		1.839	0.8969	0.9451	0.0482	7.887	8.812
2		1.773	0.8730	0.9241	0.0511	7.465	8.397
3		1.765	0.8700	0.9144	0.0444	7.413	8.211
4		1.375	0.7075	0.7207	0.0132	5.099	5.256
5		1.363	0.7018	0.7329	0.0311	5.033	5.407
6		1.362	0.7013	0.7190	0.0177	5.027	5.236
7		1.342	0.6917	0.7270	0.0353	4.917	5.334
8		1.326	0.6837	0.7404	0.0567	4.827	5.501
9		1.322	0.6818	0.7198	0.0380	4.806	5.246
10		1.295	0.6685	0.6904	0.0219	4.661	4.902
11		1.284	0.6629	0.6970	0.0341	4.602	4.978
12		1.269	0.6553	0.6858	0.0305	4.522	4.851
13		1.265	0.6531	0.6820	0.0289	4.499	4.808
14		1.221	0.6300	0.6606	0.0306	4.266	4.577
15		1.196	0.6165	0.6373	0.0208	4.135	4.338
16		1.183	0.6095	0.6418	0.0323	4.069	4.383
17		1.158	0.5955	0.6232	0.0277	3.940	4.200
18		1.130	0.5796	0.5968	0.0172	3.798	3.952
19		1.099	0.5615	0.5809	0.0194	3.643	3.810
20		1.088	0.5549	0.5767	0.0218	3.588	3.773
21		1.061	0.5385	0.5556	0.0171	3.455	3.594
22		1.060	0.5379	0.5530	0.0151	3.451	3.573
23		1.059	0.5373	0.5588	0.0215	3.446	3.621
24		1.058	0.5367	0.5598	0.0231	3.441	3.629
25		1.049	0.5312	0.5563	0.0251	3.398	3.600
26		1.048	0.5306	0.5667	0.0361	3.393	3.687
27		0.996	0.4975	0.5307	0.0332	3.144	3.394
28		0.972	0.4815	0.5139	0.0324	3.030	3.265
29		0.960	0.4735	0.5072	0.0337	2.975	3.215
30		0.940	0.4597	0.4794	0.0152	2.882	2.985
31		0.937	0.4576	0.4747	0.0171	2.868	2.983
32		0.935	0.4562	0.4758	0.0196	2.859	2.991
33		0.935	0.4561	0.4730	0.0169	2.858	2.972
34		0.933	0.4549	0.4800	0.0251	2.850	3.020
35		0.933	0.4549	0.4733	0.0184	2.850	2.974
36		0.933	0.4549	0.4716	0.0167	2.850	2.962
37		0.932	0.4541	0.4735	0.0194	2.845	2.975
38		0.925	0.4492	0.4842	0.0350	2.813	3.049
39		0.925	0.4491	0.4816	0.0325	2.813	3.031
40		0.917	0.4436	0.4706	0.0270	2.777	2.955
41		0.885	0.4203	0.4729	0.0526	2.632	2.971
42		0.861	0.4025	0.4284	0.0259	2.526	2.682
43		0.792	0.3481	0.3738	0.0257	2.229	2.365
44		0.791	0.3473	0.3718	0.0245	2.225	2.354
45		0.725	0.2905	0.2438	−0.0467	1.952	1.753
46		0.713	0.2797	0.1967	−0.0830	1.904	1.573
47		0.684	0.2526	0.2819	0.0293	1.789	1.914
48		0.645	0.2144	0.2383	0.0239	1.638	1.731
49		0.633	0.2021	0.2450	0.0429	1.593	1.758
50		0.619	0.1876	0.2068	0.0192	1.540	1.610
51		0.604	0.1715	0.1738	0.0023	1.484	1.492
52		0.582	0.1474	0.1816	0.0342	1.404	1.519
53		0.551	0.1118	0.1386	0.0268	1.294	1.376
54		0.539	0.0974	0.1255	0.0281	1.251	1.335
55		0.535	0.0926	0.1345	0.0419	1.238	1.363
56		0.524	0.0790	0.1143	0.0353	1.200	1.301
57		0.496	0.0433	0.0748	0.0315	1.105	1.188
58		0.470	0.0081	0.0224	0.0163	1.019	1.053
59		0.450	−0.0202	0.0000	0.0202	0.955	1.000
60		0.415	−0.0730	0.0310	0.1040	0.845	1.074
61		0.365	−0.1566	−0.1403	0.0163	0.697	0.724
62		0.364	−0.1583	−0.0794	0.0789	0.695	0.833
63		0.341	−0.2008	−0.1637	0.0371	0.630	0.686
64		0.333	−0.2167	−0.1931	0.0236	0.607	0.641
65		0.320	−0.2424	−0.2175	0.0249	0.572	0.606
66		0.210	−0.5167	−0.4737	0.0430	0.305	0.336
67		0.192	−0.5750	−0.3747	0.2003	0.266	0.422
68		0.129	−0.8341	−0.7986	0.0355	0.146	0.159

TABLE 23.—SHARP-CRESTED WEIRS.

Experiments on the Utah Weir, No. 3; Height, 1.64 ft.; Length, 6.53 ft.

(1) No. of exp.	(2) Head.	(3) 15-ft. tape head.	(4) Log. Q_c	(5) Log. Q_m	(6) Log. Q_m —Log. Q_c	(7) Q_c	(8) Q_m
1	0.936		0.4570	0.4749	0.0179	2.864	2.985
2	0.753		0.3152	0.3778	0.0626	2.066	2.387
3	0.750		0.3126	0.4067	0.0941	2.007	2.551
4	0.748		0.3108	0.3729	0.0621	1.999	2.360
5	0.735		0.2994	0.3652	0.0658	1.993	2.319
6	0.725		0.2904	0.3582	0.0678	1.952	2.282
7	0.723		0.2886	0.3569	0.0683	1.944	2.275
8	0.715		0.2814	0.3472	0.0658	1.912	2.224
9	0.658		0.2273	0.2892	0.0619	1.688	1.946
10	0.651		0.2204	0.2717	0.0513	1.661	1.869
11	0.614		0.1823	0.2450	0.0627	1.522	1.758
12	0.612		0.1802	0.2300	0.0502	1.514	1.698
13	0.591		0.1563	0.2157	0.0594	1.433	1.643
14	0.574		0.1383	0.1732	0.0349	1.375	1.490
15	0.568		0.1315	0.1798	0.0483	1.354	1.513
16	0.548		0.1082	0.1657	0.0575	1.283	1.465
17	0.544		0.1034	0.1598	0.0564	1.269	1.445
18	0.543		0.1021	0.1541	0.0520	1.267	1.426
19	0.540		0.0986	0.1383	0.0397	1.255	1.375
20	0.534		0.0913	0.1345	0.0432	1.234	1.363
21	0.530		0.0865	0.1532	0.0667	1.220	1.423
22	0.527		0.0827	0.1245	0.0418	1.210	1.332
23	0.525		0.0803	0.1298	0.0495	1.203	1.348
24	0.524		0.0790	0.1169	0.0379	1.200	1.309
25	0.523		0.0777	0.1273	0.0496	1.196	1.341
26	0.512		0.0640	0.1186	0.0546	1.159	1.314
27	0.502		0.0510	0.0752	0.0242	1.125	1.189
28	0.502		0 0510	0.0224	—0.0286	1.125	1.053
29	0.499		0.0471	0.1005	0.0534	1.115	1.260
30	0.494		0.0406	0.0748	0.0342	1.098	1.188
31	0.491		0.0366	0.0934	0.0568	1.088	1.240
32	0.486		0.0299	0.0759	0.0460	1.071	1.191
33	0.478		0.0191	0.0700	0.0509	1.045	1.175
34	0.476		0.0164	0.0715	0.0551	1.038	1.179
35	0.476		0.0164	0.0713	0.0549	1.038	1.178
36	0.474		0.0137	0.0853	0.0716	1.032	1.217
37	0.471		0.0095	—0.0414	—0.0509	1.022	0.909
38	0.468		0.0053	—0.0685	—0.0738	1.012	0.854
39	0.468		0.0053	0.0531	0.0478	1.012	1.130
40	0.460		—0.0058	0.0310	0.0368	0.987	1.074
41	0.459		—0.0073	—0.1652	—0.1570	0.983	0.683
42	0.458		—0.0086	0.0406	0.0492	0.980	1.098
43	0.456		—0.0115	0.0374	0.0489	0.974	1.090
44	0.454		—0.0144	0.0269	0.0413	0.967	1.064
45	0.450		—0.0202	0.1614	0.1816	0.955	1.450
46	0.445		—0.0274	0.0319	0.0593	0.939	1.076
47	0.442		—0.0319	0.0216	0.0535	0.929	1.051
48	0.441		—0.0334	0.0175	0.0509	0.926	1.041
49	0.440		—0.0348	0.0000	0.0384	0.923	1.000
50	0.435		—0.0422	0.0000	0.0422	0.907	1.000
51	0.434		—0.0438	—0.0004	0.0434	0.904	0.999
52	0.431		—0.0482	0.0004	0.0486	0.895	1.001
53	0.431		—0.0482	—0.0074	0.0408	0.895	0.983
54	0.429		—0.0512	0.0009	0.0521	0.889	1.002
55	0.427		—0.0544	—0.0031	0.0513	0.882	0.993
56	0.403		—0.0920	—0.0353	0.0567	0.809	0.922
57	0.399		—0.0985	—0.0560	0.0425	0.797	0.879
58	0.396		—0.1035	—0.0533	0.0502	0.788	0.885
59	0.383		—0.1252	—0.0721	0.0531	0.750	0.847
60	0.383		—0.1252	—0.0942	0.0310	0.750	0.805
61	0.381		—0.1286	—0.0841	0.0445	0.744	0.824
62	0.381		—0.1286	—0.0846	0.0440	0.744	0.823
63	0.381		—0.1286	—0.0862	0.0424	0.744	0.820
64	0.381		—0.1286	—0.0878	0.0408	0.744	0.817

TABLE 23.—(*Continued.*)

(1) No. of exp.	(2) Head.	(3) 15-ft. tape head.	(4) Log. Q_c	(5) Log. Q_m	(6) Log. Q_m —Log. Q_c	(7) Q_c	(8) Q_m
65	0.380		—0.1303	—0.0585	0.0718	0.741	0.874
66	0.380		—0.1303	—0.0794	0.0609	0.741	0.833
67	0.380		—0.1303	—0.0846	0.0457	0.741	0.823
68	0.380		—0.1303	—0.0862	0.0441	0.741	0.820
69	0.379		—0.1321	—0.0904	0.0417	0.738	0.812
70	0.379		—0.1321	—0.0857	0.0464	0.738	0.821
71	0.377		—0.1356	—0.0794	0.0562	0.732	0.833
72	0.376		—0.1372	—0.0894	0.0478	0.729	0.814
73	0.374		—0.1407	—0.0947	0.0460	0.723	0.804
74	0.373		—0.1425	—0.0856	0.0569	0.720	0.821
75	0.371		—0.1459	—0.0814	0.0645	0.715	0.829
76	0.357		—0.1710	—0.0944	0.0766	0.675	0.805
77	0.348		—0.1876	—0.1403	0.0473	0.650	0.724
78	0.347		—0.1896	—0.1343	0.0553	0.646	0.734
79	0.346		—0.1914	—0.1388	0.0526	0.644	0.726
80	0.345		—0.1933	—0.1451	0.0482	0.641	0.716
81	0.345		—0.1933	—0.1637	0.0296	0.641	0.686
82	0.338		—0.2076	—0.1701	0.0375	0.621	0.676
83	0.333		—0.2164	—0.1657	0.0507	0.608	0.683
84	0.328		—0.2262	—0.1931	0.0331	0.594	0.641
85	0.326		—0.2302	—0.1931	0.0371	0.589	0.641
86	0.318		—0.2464	—0.2175	0.0289	0.567	0.606
87	0.314		—0.2546	—0.1918	0.0628	0.556	0.643
88	0.310		—0.2629	—0.2056	0.0573	0.546	0.623
89	0.310		—0.2629	—0.2060	0.0569	0.546	0.622
90	0.301		—0.2821	—0.2311	0.0510	0.522	0.587
91	0.294		—0.2976	—0.2476	0.0500	0.504	0.565
92	0.288		—0.3109	—0.2636	0.0473	0.489	0.545
93	0.278		—0.3340	—0.2882	0.0458	0.463	0.515
94	0.276		—0.3387	—0.2853	0.0534	0.458	0.518
95	0.273		—0.3451	—0.2880	0.0571	0.452	0.515
96	0.270		—0.3529	—0.2967	0.0562	0.444	0.505
97	0.256		—0.3877	—0.3279	0.0598	0.410	0.470
98	0.247		—0.4110	—0.3675	0.0435	0.388	0.429
99	0.245		—0.4162	—0.3625	0.0537	0.384	0.434
100	0.243		—0.4216	—0.3526	0.0690	0.379	0.444
101	0.233		—0.4489	—0.3840	0.0649	0.356	0.413
102	0.229		—0.4603	—0.3925	0.0678	0.346	0.405
103	0.228		—0.4633	—0.4021	0.0612	0.344	0.396
104	0.221		—0.4834	—0.4286	0.0548	0.328	0.373
105	0.217		—0.4952	—0.4437	0.0515	0.320	0.360
106	0.215		—0.5014	—0.4457	0.0557	0.315	0.358
107	0.213		—0.5074	—0.4737	0.0337	0.311	0.336
108	0.213		—0.5074	—0.4634	0.0440	0.311	0.344
109	0.210		—0.5176	—0.4529	0.0638	0.304	0.352
110	0.208		—0.5229	—0.3747	0.1482	0.300	0.422
111	0.205		—0.5323	—0.4816	0.0507	0.294	0.330
112	0.203		—0.5388	—0.4929	0.0459	0.289	0.321
113	0.201		—0.5452	—0.4930	0.0522	0.285	0.321
114	0.186		—0.5951	—0.5257	0.0694	0.254	0.298
115	0.180		—0.6170	—0.5768	0.0402	0.242	0.265
116	0.174		—0.6393	—0.5953	0.0443	0.229	0.254
117	0.173		—0.6430	—0.5931	0.0499	0.228	0.255
118	0.173		—0.6430	—0.6044	0.0386	0.228	0.249
119	0.152		—0.7273	—0.6882	0.0391	0.187	0.205
120	0.132		—0.8131	—0.5567	0.2624	0.154	0.278
121	0.131		—0.8240	—0.7986	0.0254	0.150	0.159
122	0.119		—0.8867	—0.8356	0.0511	0.130	0.146
123	0.115		—0.9090	—0.8446	0.0644	0.123	0.143
124	0.115		—0.9090	—0.8446	0.0644	0.123	0.143
125	0.109		—0.9439	—0.8758	0.0681	0.114	0.133
126	0.103		—0.9808	—0.9051	0.0757	0.104	0.124
127	0.087		—1.0907	—1.0223	0.0684	0.081	0.095
128	0.065		—1.2807	—1.1634	0.1173	0.052	0.069
129	0.062		—1.3114	—1.1935	0.1179	0.049	0.064
130	0.048		—1.4782	—1.3872	0.0910	0.033	0.041
131	0.041		—1.5808	—1.5229	0.0579	0.026	0.030

TABLE A.—HEADS ON THE CORNELL STANDARD WEIR (HEIGHT 11.25 FT.) AS MEASURED SIMULTANEOUSLY BY DIFFERENT METHODS.

All Heads are in Centimeters. (1 cm. = 0.0328 ft.)

No.	Date, 1901.	Standard tube piezometer.	6-ft. longitudinal piezometer.	6-ft. wall piezometer.	6-ft. tape.	11-ft. wall piezometer.	2-ft. wall piezometer.	Crest piezometer.	Point gauge at crest (bent).	Point gauge at crest.
1	May 15	9.146	9.356	9.296	....	9.268	9.306	....	....	....
2	"	53.594	53.622	53.748	....	53.766	52.192	....	....	....
3	"	53.336	53.316	53.442	....	53.448	51.854	....	....	....
4	"	53.316	53.302	53.412	....	53.474	51.928	....	....	....
5	"	94.076	94.668	94.652	....	95.488	90.022	....	....	....
6	"	94.844	95.102	95.086	....	96.756	90.072	....	81.95	....
7	"	94.686	....	....	....	....	....	....	81.80	....
8	"	42.282	42.580	42.654	....	42.538	41.678	....	....	....
9	May 16	45.480	45.840	45.730	45.75	45.564	44.520	30.350	38.42	....
10	"	77.846	77.904	78.176	78.24	78.126	74.860	52.100	66.86	....
11	"	69.508	70.140	70.392	70.35	70.230	67.510	46.820	59.54	....
12	"	59.964	59.938	59.994	59.89	60.166	57.908	40.210	50.44	....
13	"	54.048	54.080	54.244	53.98	54.224	52.520	36.172	....	....
14	"	47.932	47.998	48.194	48.10	48.012	46.888	32.048	40.30	....
15	"	36.274	36.468	36.598	36.30	36.344	35.774	23.694	30.28	....
16	"	28.348	28.168	28.184	28.04	27.990	27.788	17.728	23.54	....
17	May 17	22.372	....	22.464	22.43	22.468	22.094	....	18.53	....
18	"	18.822	19.086	18.924	18.96	18.900	18.804	10.900	15.51	....
19	"	16.492	16.644	16.536	16.58	16.544	16.502	9.968	13.51	....
20	"	13.876	14.020	13.940	14.02	13.946	13.918	8.378	....	....
21	"	13.898	14.044	13.942	14.02	13.962	13.924	8.398	11.13	....
22	"	11.204	11.422	11.316	11.40	11.344	11.290	7.210	....	....
23	"	11.292	11.426	11.322	11.40	11.388	11.300	7.206	9.08	....
24	"	7.628	7.808	7.718	7.74	7.700	7.718	4.822	5.87	....
25	"	39.298	39.454	39.416	39.44	41.406	38.684	25.242	32.98	....
26	May 18	21.082	....	....	....	....	....	....	....	....
27	May 20	....	....	....	....	....	....	....	....	....
28	"	21.240	21.430	21.268	21.28	21.274	21.088	13.208	17.55	....
29	"	27.260	27.452	27.288	27.31	27.290	27.054	17.242	22.77	....
30	"	32.752	33.010	32.814	32.89	32.842	32.386	21.036	27.30	....
31	"	50.372	50.586	50.410	50.23	50.530	49.016	32.980	42.34	....
32	"	61.830	62.026	61.852	61.45	62.024	59.768	40.602	52.19	....
33	"	66.872	67.146	66.880	66.78	....	64.670	44.204	56.29	....
34	"	73.434	73.486	73.418	73.46	....	70.474	48.578	62.22	....
35	May 23	16.672	16.734	16.810	16.61	16.672	16.642	10.150	13.65	....
36	"	97.530	97.406	98.034	96.56	....	91.872	65.234	83.65	....
37	"	90.566	90.630	90.802	89.89	91.152	86.038	61.354	77.15	....
38	"	85.628	85.584	85.902	84.46	86.138	81.598	57.850	73.85	....
39	"	81.548	81.822	82.018	81.32	81.732	78.338	55.258	69.45	....
40	"	64.910	65.066	64.966	64.89	65.134	62.594	43.048	54.65	....
41	"	43.884	43.932	43.856	43.13	43.914	42.864	28.860	36.35	....
42	May 29	44.978	45.108	44.918	....	....	44.094	29.574	....	87.83
43	"	44.934	45.128	....	....	....	43.972	29.648	....	87.79
44	"	44.878	45.160	44.938	....	....	43.958	29.630	....	....
45	"	69.076	69.810	69.112	....	69.366	66.512	45.750	....	....
46	"	68.966	69.260	....	....	69.198	66.440	45.708	....	58.43
47	"	115.836	116.306	116.432	....	116.748	108.336	81.614	....	....
48	"	116.816	116.922	116.676	....	117.152	108.718	82.308	....	....
49	"	105.444	105.870	105.054	....	106.018	99.038	72.188	....	....
50	"	105.652	105.860	104.616	....	106.218	99.124	71.958	....	....
51	"	91.198	91.376	....	....	91.528	87.110	60.244	....	....
52	"	73.972	74.312	73.078	....	73.934	71.590	49.928	....	....
53	May 30	....	....	....	....	74.300	....	....	....	....
54	"	....	....	....	....	....	....	....	....	....
55	"	73.936	....	74.206	....	....	71.250	....	....	....
56	"	73.922	74.282	....	....	....	....	50.590	....	....
57	"	73.788	....	....	....	....	....	....	....	....
58	June 1	111.338	111.436	111.282	....	111.588	104.336	77.270	....	95.03
59	"	111.524	111.250	111.542	....	111.668	104.282	77.380	....	94.83
60	"	111.500	111.150	111.082	....	111.938	103.844	76.988	....	95.20

TABLE A.—(*Continued.*)

No.	Date, 1901.	Standard tube piezometer.	6-ft. longitudinal piezometer.	6-ft. wall piezometer.	6-ft. tape.	11-ft. wall piezometer.	2-ft. wall piezometer.	Crest piezometer.	Point gauge at crest (bent).	Point gauge at crest.
61	June 1	111.220	110.454	110.800	....	111.594	103.614	77.644	....	95.27
62	"	111.808	110.744	111.956	....	111.846	103.754	77.452	....	95.10
63	"	111.648	111.158	111.404	....	....	104.056	77.976	....	....
64	"	111.458	....	....	....	....	....	....	....	....
65	"	110.316	....	....	....	....	....	....	....	....
66	June 3	....	....	....	....	....	....	....	....	....
67	"	101.908	102.196	102.100	....	102.448	96.036	68.626	....	87.72
68	"	102.218	102.104	102.148	....	102.762	96.406	68.754	....	87.47
69	"	102.264	102.468	102.436	....	102.688	96.664	49.832	....	87.51
70	"	92.968	92.848	93.254	....	93.142	88.182	62.328	....	80.35
71	"	92.700	92.612	92.800	....	93.236	88.100	62.872	....	79.84
72	"	92.716	92.784	92.830	....	93.014	88.100	62.938	....	80.08
73	"	92.606	92.438	92.724	....	92.834	78.878	62.664	....	79.89
74	"	92.850	92.750	93.238	....	93.238	88.182	63.002	....	80.15
75	"	84.248	84.060	84.356	....	84.560	80.662	56.270	....	72.45
76	"	84.356	84.122	84.456	....	84.756	80.478	56.308	....	72.45
77	"	84.250	84.122	83.382	....	84.592	80.538	55.696	....	72.46
78	June 6	29.942	....	....	29.75	....	....	....	....	....
79	"	30.002	....	....	29.72	....	....	....	....	....
80	"	97.046	....	....	96.71	....	....	....	....	....
81	"	96.400	95.962	96.790	96.38	....	91.488	65.390	....	....
82	"	86.738	86.876	86.976	86.50	....	82.710	58.282	....	....
83	"	87.148	87.118	87.372	86.59	....	83.028	58.458	....	....
84	"	87.094	86.956	87.138	86.50	....	82.832	58.622	....	....
85	"	77.580	77.416	77.584	77.05	....	74.348	51.852	....	....
86	"	64.800	64.866	64.744	64.34	....	62.428	42.846	....	....
87	"	53.748	53.906	53.884	53.37	....	52.172	35.680	....	....
88	"	46.114	46.220	46.240	45.69	....	45.188	30.546	....	....
89	"	32.880	32.886	32.764	32.61	....	32.364	20.992	....	....
90	June 11	31.676	....	....	....	....	....	....	....	....
91	"	33.616	33.696	33.694	....	....	33.186	22.262	....	....
92	"	79.288	79.106	79.328	....	....	75.872	52.996	....	....
93	"	78.538	78.590	78.660	....	....	75.098	52.816	....	....
94	"	78.312	78.502	79.206	....	....	75.878	52.858	....	....
95	"	61.192	61.254	61.138	....	....	59.106	40.642	....	....
96	"	61.502	61.496	61.554	....	....	59.400	40.698	....	....
97	"	52.792	52.824	52.810	....	....	51.330	34.942	....	....
98	"	52.818	52.782	52.750	....	....	51.292	35.020	....	....
99	"	52.792	52.800	52.850	....	....	51.334	35.100	....	....
100	"	69.732	70.116	69.448	....	....	67.630	46.972	....	....
101	"	69.792	70.020	69.874	....	....	67.896	46.784	....	....
102	"	69.578	69.692	69.848	....	....	67.246	46.708	....	....
103	June 14	54.994	54.940	54.862	....	....	....	....	....	....
104	"	54.984	54.906	54.892	....	....	....	....	....	....
105	"	54.954	54.878	54.864	....	....	....	....	....	....
106	"	54.882	54.852	54.764	....	....	....	....	....	....
107	"	44.950	....	....	....	....	....	....	....	....
108	"	54.954	....	....	....	....	....	....	....	....
109	June 17	....	....	....	....	....	....	....	....	....
110	"	2.006	....	....	....	....	....	....	....	....
111	July 22	29.850	....	....	....	....	....	18.396	....	....
112	"	29.842	....	....	....	....	....	18.104	....	....
113	"	27.268	....	....	....	....	....	16.744	....	....
114	"	22.646	....	....	....	....	....	13.912	....	....
115	"	17.638	....	....	....	....	....	10.866	....	....
116	"	14.664	....	....	....	....	....	9.018	....	....
117	"	9.306	....	....	....	....	....	5.234	....	....
118	"	6.022	....	....	....	....	....	3.602	....	....
119	"	6.030	....	....	....	....	....	3.618	....	....

TABLE A.—(*Continued.*)

HEADS ON THE CORNELL STANDARD WEIR (HEIGHT 6.65 FT.), AS MEASURED SIMULTANEOUSLY BY DIFFERENT METHODS. THE READINGS TAKEN SIMULTANEOUSLY ON THE STANDARD TUBE-PIEZOMETER OF THE CORRESPONDING HEADS ON THE CORNELL STANDARD WEIR (HEIGHT 11.25 FT.) ARE ALSO GIVEN IN THIS TABLE.

All Heads are in Centimeters. (1 cm. = 0.0328 ft.)

No.	Date, 1901.	Standard tube piezometer.	6-ft. Francis piezometer.	6-ft. bottom piezometer.	6-ft. float piezometer.	6-ft. point gauge.	10-ft. tape.	13-ft. three-tube piezometer.	14-ft. tape.	16-ft. one tube piezometer.	Point gauge at crest.
1	May 15	9.146	8.802	8.800	......	......	......	......	......	......	......
2	"	53.594	53.202	53.400	......	......	......	......	......	......	......
3	"	53.336	52.878	53.000	......	......	......	......	......	......	......
4	"	53.316	52.920	53.100	......	......	......	......	......	......	......
5	"	94.076	92.636	93.800	......	......	......	......	......	......	......
6	"	94.844	92.500	93.600	......	......	......	......	......	......	......
7	"	94.686	92.454	93.600	......	......	......	......	......	......	......
8	"	42.282	42.176	42.800	......	......	......	......	......	......	......
9	May 16	45.480	45.223	45.600	......	......	......	......	......	......	......
10	"	77.846	77.224	77.600	......	......	......	......	......	......	......
11	"	69.508	69.449	70.100	......	......	......	......	......	......	......
12	"	59.964	59.117	60.000	......	......	......	......	......	......	......
13	"	54.048	53.723	54.200	......	......	......	......	......	......	......
14	"	47.932	47.881	48.100	......	......	......	......	......	......	......
15	"	36.274	36.117	36.400	......	......	......	......	......	......	......
16	"	28.348	28.178	28.200	......	......	......	......	......	......	......
17	May 17	22.872	22.249	22.200	......	......	......	......	......	......	......
18	"	18.822	18.787	18.700	......	......	......	......	......	......	......
19	"	16.492	16.395	16.400	......	......	......	......	......	......	......
20	"	13.876	13.797	13.740	......	......	......	......	......	......	......
21	"	13.938	13.773	13.800	......	......	......	......	......	......	......
22	"	11.204	11.117	11.100	......	......	......	......	......	......	......
23	"	11.292	11.100	11.050	......	......	......	......	......	......	......
24	"	7.628	7.619	7.570	......	......	......	......	......	......	......
25	"	39.298	39.046	39.090	......	......	......	......	......	......	......
26	May 18	21.082	20.890	20.900	......	......	......	......	......	......	......
27	May 20	......	......	......	......	......	......	......	......	......	......
28	"	21.240	21.272	21.300	......	......	......	......	......	......	......
29	"	27.260	27.264	27.300	......	......	......	......	......	......	......
30	"	32.752	32.690	32.800	......	......	......	......	......	......	......
31	"	50.372	50.074	50.200	......	......	......	......	......	......	......
32	"	61.830	61.346	61.700	......	......	......	......	......	......	......
33	"	66.872	66.478	......	......	......	......	......	......	......	......
34	"	73.434	72.887	73.500	......	......	......	......	......	......	......
35	May 23	16.672	16.417	16.500	......	......	......	......	......	......	......
36	"	99.530	95.067	95.870	......	......	......	......	......	......	......
37	"	90.566	88.417	89.360	......	......	......	......	......	......	......
38	"	85.628	83.147	84.010	......	......	......	......	......	......	......
39	"	81.548	79.669	80.460	......	......	......	......	......	......	......
40	"	64.910	63.627	64.390	......	......	......	......	......	......	......
41	"	43.884	42.757	42.840	......	......	......	......	......	......	......
42	May 29	44.978	44.687	44.860	......	......	......	......	......	......	......
43	"	44.934	44.803	44.840	......	......	......	......	......	......	......
44	"	44.878	44.577	44.670	......	......	......	......	......	......	......
45	"	69.076	68.421	68.940	......	......	......	......	......	......	......
46	"	68.966	67.874	68.360	......	......	......	......	......	......	......
47	"	115.836	113.835	116.020	......	......	......	......	......	......	......
48	"	116.316	116.252	118.570	......	......	......	......	......	......	......
49	"	105.444	103.602	104.360	......	......	......	......	......	......	......
50	"	105.652	103.567	105.380	......	......	......	......	......	......	......
51	"	91.198	89.417	90.660	......	......	......	......	......	......	......
52	"	73.972	73.182	74.100	......	......	......	......	......	......	......
53	May 30	......	......	......	......	......	......	......	......	......	......

TABLE A.—(*Continued.*)

No.	Date, 1901.	Standard tube piezometer.	6 ft. Francis piezometer.	6-ft. bottom piezometer.	6-ft. float piezometer.	6-ft. point gauge.	10-ft. tape.	13-ft. three-tube piezometer.	14-ft. tape.	16-ft. one-tube piezometer.	Point gauge at crest.
54	May 30	......	......	......	......	......	......	......	......	......	......
55	“	73.936	......	......	......	......	......	......	......	......	......
56	“	73.922	......	......	......	......	......	......	......	......	......
57	“	73.788	72.927	73.730	......	......	......	......	......	......	......
58	June 1	111.338	108.912	110.670	......	......	......	......	......	108.428	......
59	“	111.524	108.917	110.550	......	......	109.45	......	......	108.942	......
60	“	111.500	109.199	110.830	......	......	109.15	......	......	109.088	......
61	“	111.220	108.537	110.100	......	......	108.08	......	......	108.122	......
62	“	111.308	107.856	109.660	......	......	108.17	......	......	108.212	......
63	“	111.648	108.276	109.850	......	......	......	......	......	108.568	......
64	“	111.458	108.637	110.360	......	......	......	......	......	108.680	......
65	“	110.316	108.435	110.220	......	......	......	......	......	108.576	......
66	June 3	......	......	......	......	......	......	......	......	31.760	......
67	“	101.908	99.097	100.510	......	......	99.82	......	......	99.70	84.82
68	“	102.218	99.878	101.870	......	99.24	100.43	......	......	100.44	......
69	“	102.264	99.907	101.330	......	......	100.13	......	......	100.50	85.86
70	“	92.968	90.628	92.010	......	......	90.92	......	......	91.28	77.79
71	“	92.700	90.423	91.690	......	......	90.89	......	......	91.18	......
72	“	92.716	90.745	92.040	......	......	91.10	......	......	91.50	......
73	“	92.606	90.745	91.900	......	90.65	90.95	......	......	91.46	......
74	“	92.850	90.678	91.820	......	......	90.98	......	......	91.24	......
75	“	84.248	82.420	83.410	......	......	82.97	......	......	83.32	70.93
76	“	84.356	82.577	83.580	......	82.30	82.97	......	......	83.42	......
77	“	84.250	83.011	83.920	......	82.51	83.18	......	......	83.742	71.29
78	June 6	29.942	29.740	......	29.85	......	29.72	......	......	29.504	25.43
79	“	30.002	29.678	......	29.85	30.05	29.90	......	......	29.528	......
80	“	97.046	94.064	......	95.25	......	95.16	......	......	94.344	81.20
81	“	96.400	93.450	......	94.82	94.00	94.69	......	......	94.164	......
82	“	86.738	84.450	......	85.50	......	84.43	......	......	85.212	73.00
83	“	87.148	84.850	......	85.95	84.86	84.83	......	......	85.526	......
84	“	87.094	84.688	......	85.62	......	84.80	......	......	85.344	......
85	“	77.580	75.981	......	76.83	76.50	76.05	......	......	76.940	65.93
86	“	64.800	63.850	......	64.39	64.22	63.89	......	......	64.330	......
87	“	53.748	52.983	......	53.34	53.34	53.55	......	......	54.152	45.60
88	“	46.114	45.542	......	45.78	45.87	46.06	......	......	46.792	39.29
89	“	32.880	32.526	......	32.56	32.83	32.71	......	......	33.944	27.86
90	June 11	31.676	......	......	......	......	......	......	......	......	......
91	“	33.616	33.546	......	33.65	......	33.07	......	......	34.290	29.55
92	“	79.288	78.036	......	78.74	78.24	78.18	......	......	78.162	66.96
93	“	78.538	77.327	......	77.97	76.35	77.24	......	......	77.480	66.49
94	“	78.312	77.297	......	78.07	......	77.58	......	......	77.416	65.77
95	“	61.192	60.703	......	61.17	......	60.90	......	......	61.140	51.69
96	“	61.502	60.922	......	61.37	60.56	60.93	......	......	61.384	51.97
97	“	52.792	52.437	......	52.77	......	52.36	......	......	52.354	......
98	“	52.818	52.382	......	52.70	......	......	......	......	52.352	44.44
99	“	52.792	52.365	......	52.70	52.42	52.27	......	......	52.370	......
100	“	69.732	69.384	......	70.00	69.25	69.37	......	......	69.646	59.13
101	“	69.792	69.123	......	69.80	......	69.17	......	......	69.390	59.44
102	“	69.578	69.032	......	69.67	......	69.04	......	......	69.228	......
103	June 14	54.994	......	......	......	......	......	......	......	......	......
104	“	54.984	......	......	......	......	......	......	......	......	......
105	“	54.954	......	......	......	......	......	......	......	......	......
106	“	54.882	......	......	......	......	......	......	......	......	......
107	“	44.950	54.299	......	54.64	......	......	......	......	55.172	......
108	“	54.954	54.929	......	55.30	......	......	......	......	55.196	......
109	June 17	......	......	......	......	......	......	......	......	......	......
110	“	2.006	1.827	......	1.83	......	......	......	......	......	......
111	July 22	29.850	......	......	......	......	29.65	......	......	29.764	......
112	“	29.842	......	......	......	......	29.63	......	......	29.722	......
113	“	27.268	......	......	......	......	27.03	......	......	27.168	......
114	“	22.646	22.522	......	22.584	......	22.26	......	......	22.482	......
115	“	17.638	17.618	......	17.586	......	17.35	......	......	17.476	......
116	“	14.664	14.621	......	14.654	......	14.62	......	......	14.512	......
117	“	9.306	9.306	......	9.202	......	8.998	......	......	9.176	......

TABLE A.—(*Continued.*)

No.	Date, 1901.	Standard tube piezometer.	6-ft. Francis piezometer.	6-ft. bottom piezometer.	6-ft. float piezometer.	6-ft. point gauge.	10-ft. tape.	13-ft. three-tube piezometer.	14-ft. tape.	16-ft. one-tube piezometer.	Point gauge at crest.
118	July 22	6.022	6.107	......	6.149	......	5.936	......	......	5.762	......
119	"	6.030	5.960	......	6.013	......	5.727	......	......	5.876	......
120	Oct. 22	31.912	......	......	......	......	......	31.562	31.948	......	......
121	"	31.826	......	......	......	......	......	31.542	31.618	......	......
122	"	26.400	......	......	......	......	......	26.218	25.642	......	......
123	"	22.788	......	......	......	......	......	22.532	......	......	......
124	"	22.750	......	......	......	......	......	22.534	22.806	......	......
125	"	10.610	......	......	......	......	......	10.586	10.610	......	......
126	Oct. 25	36.898	......	......	......	......	......	36.526	......	......	......
127	"	36.796	......	......	......	......	......	36.506	......	......	......
128	"	36.664	......	......	......	......	......	36.392	......	......	......
129	"	28.566	......	......	......	......	......	28.364	......	......	......
130	"	28.490	......	......	......	......	......	28.228	......	......	......
131	"	34.052	......	......	......	......	......	33.736	......	......	......
132	"	33.914	......	......	......	......	......	33.708	......	......	......
133	"	40.088	......	......	......	......	......	39.766	......	......	......
134	"	43.930	......	......	......	......	......	43.630	......	......	......
135	"	46.798	......	......	......	......	......	46.710	......	......	......
136	"	50.796	......	......	......	......	......	50.376	......	......	......
137	"	41.730	......	......	......	......	......	41.462	......	......	......
138	Oct. 29	5.024	......	......	......	......	......	4.754	4.818	......	......
139	"	5.048	......	......	......	......	......	4.736	4.818	......	......
140	"	49.024	......	......	......	......	......	48.518	48.602	......	......
141	"	42.406	......	......	......	......	......	41.890	41.816	......	......
142	"	33.830	......	......	......	......	......	33.424	33.600	......	......
143	"	28.964	......	......	......	......	......	28.620	28.782	......	......
144	"	23.946	......	......	......	......	......	23.564	23.568	......	......
145	"	20.612	......	......	......	......	......	20.284	20.306	......	......
146	"	17.004	......	......	......	......	......	16.678	16.770	......	......
147	"	12.638	......	......	......	......	......	12.586	12.744	......	......
148	"	7.648	......	......	......	......	......	7.274	......	......	......
149	Nov. 1	75.360	......	......	......	......	......	74.590	74.914	......	......
150	"	75.140	......	......	......	......	......	74.612	74.488	......	......
151	"	67.234	......	......	......	......	......	66.526	66.774	......	......
152	"	60.990	......	......	......	......	......	60.188	60.248	......	......
153	"	54.250	......	......	......	......	......	51.572	53.692	......	......
154	"	50.710	......	......	......	......	......	49.858	50.124	......	......
155	"	45.182	......	......	......	......	......	44.620	44.606	......	......
156	"	45.990	......	......	......	......	......	44.414	44.794	......	......
157	"	10.374	......	......	......	......	......	10.152	10.184	......	......
158	Nov. 22	16.010	......	......	......	......	......	15.676	15.672	......	......
159	"	44.146	......	......	......	......	......	43.624	43.278	......	......
160	"	89.910	......	......	......	......	......	88.128	88.178	......	......
161	"	89.996	......	......	......	......	......	88.142	86.178	......	......
162	"	81.984	......	......	......	......	......	80.432	80.586	......	......
163	"	82.046	......	......	......	......	......	80.650	80.828	......	......
164	"	71.802	......	......	......	......	......	70.714	70.676	......	......
165	"	71.526	......	......	......	......	......	70.008	70.524	......	......
166	"	22.404	......	......	......	......	......	21.960	22.074	......	......

right line is drawn. The points in the left-hand portion of the figure, through which two broken lines are drawn, show the results obtained when a correction is made for velocity of approach.

(*c*).—*Fteley and Stearns Experiments.*—The Fteley and Stearns experiments and the results of computations made on them are given in Tables 7, 13, 15, 16, 20, 26, and 28, and the plottings showing the trial equation, the adopted equations, the results of the final diagram,

TABLE A.—(*Continued.*)

Heads on the Cornell Standard Weir (Height 11.25 Ft.) as Measured Simultaneously by Different Methods.

All Heads are in Centimeters. (1 cm. = 0.0328 ft.)

No.	Date, 1902.	Standard tube-piezometer.	10-ft. tape.	No.	Date, 1902.	Standard tube-piezometer.	10-ft. tape.	No.	Date, 1902.	Standard tube-piezometer.	10-ft. tape.
84	Nov. 7	1.620	1.796	114	Nov. 13	12.322	12.166	145	Nov. 21	62.966	63.510
85	"	31.628	31.680	115	Nov. 14	47.942	......	146	"	43.452	43.386
86	"	27.432	27.410	116	"	33.372	37.412	147	"	35.836	35.856
87	"	23.270	23.172	117	"	30.924	30.826	148	"	29.510	29.454
88	"	18.136	18.142	118	"	24.624	24.452	149	"	24.956	24.850
89	"	10.354	10.336	119	"	19.814	19.606	150	Nov. 24	46.242	46.192
90	Nov. 10	31.676	31.496	120	"	12.742	12.500	151	"	37.264	37.412
91	"	26.498	26.222	121	"	31.560	31.374	152	"	30.090	30.094
92	"	22.860	22.624	122	"	24.000	23.782	153	"	23.458	23.386
93	"	18.258	18.020	124	Nov. 18	53.804	54.090	154	"	17.810	17.684
94	"	13.984	13.720	125	"	45.464	45.552	155	"	11.552	11.404
95	"	11.086	10.886	126	"	38.988	39.148	156	Nov. 25	91.420	92.538
96	"	6.240	5.976	127	"	33.162	33.234	157	"	82.700	84.122
97	Nov. 11	48.240	48.540	128	"	26.910	28.832	158	"	75.840	76.926
98	"	35.046	35.154	129	"	19.752	19.636	159	"	67.100	67.566
99	"	28.228	28.264	130	Nov. 19	61.614	62.048	160	"	59.526	59.822
100	"	21.638	21.586	131	"	48.944	49.242	161	"	53.752	54.120
101	"	14.246	14.238	132	"	38.798	38.966	162	"	44.442	44.820
102	Nov. 12	62.532	63.174	133	"	33.622	33.660	163	Nov. 26	90.430	91.896
103	"	45.768	45.978	134	"	26.784	27.258	164	"	78.072	79.366
104	"	35.618	35.734	135	"	20.350	20.214	165	"	65.680	66.316
105	"	28.670	28.570	136	"	13.308	13.202	166	"	45.706	46.070
106	"	22.516	22.196	137	Nov. 20	40.246	40.246	167	"	36.202	......
107	"	15.628	15.550	138	"	62.836	62.810	*a*	....	50.574	50.980
108	"	9.342	9.148	139	"	49.828	49.912	*b*	....	45.210	45.460
109	Nov. 13	52.334	52.474	140	"	33.968	34.026	*c*	....	39.214	39.240
110	"	39.628	39.728	141	"	27.188	27.076	*d*	....	33.024	33.112
111	"	30.416	30.460	142	"	20.328	20.092	*e*	....	28.300	28.294
112	"	23.954	23 874	143	Nov. 21	12 832	......	*f*	....	25.806	25.764
113	"	17.012	16.830	144	"	91.220	92.294				

and the degree of accuracy with which these fit the original experiments, are on Figs. 7, 13, 15, and 16, of Plate XXVII*a*, and Figs. 20, 26, and 28, of Plate XXVII*b*. The curves of these equations are brought together on Plates XXVIII and XXIX, where they, with others, are plotted on the same axes, for purposes of comparison.

The heads on the Fteley and Stearns weirs, of heights 3.56, 2.60, 1.70, 1.00, and 0.50 ft., were measured with a device similar to that used by Mr. Francis, or similar to the Cornell 6-ft. Francis piezometer,* and therefore no correction has been applied to them.

During each series of experiments on these weirs† the quantity of water flowing was kept constant; and as one of these (that with height

* *Transactions*, Am. Soc. C. E., Vol. XII, p. 9.

† *Transactions*, Am. Soc. C. E., Vol. XII, p. 54.

TABLE B.—Comparison of Heads Measured Simultaneously with the Hook-Gauge in the Bazin Pit, and with the 15-ft. Tape. These Heads Were Read on the Cornell Standard Weir of Height 6.65 Ft., and are in Centimeters. (1 cm. = 0.0328 ft.)

No. of experiment.	Bazin pit.	15-ft. tape.	No. of experiment.	Bazin pit.	15-ft. tape.	No. of experiment.	Bazin pit.	15-ft. tape.
58	1.716	1.771	77	19.064	19.126	29	45.296	44.900
23	3.242	3.251	69	19.070	19.166	100	47.736	47.713
57	3.970	4.001	52	20.384	20.461	64	48.352	47.972
25	4.058	4.068	91	20.442	20.370	44	48.976	48.926
22	5.430	5.376	103	21.248	21.260	78	51.862	51.998
56	7.230	7.235	68	23.348	23.409	87	53.786	53.535
21	8.510	8.403	76	23.614	23.676	32	58.062	57.994
55	10.110	10.290	18	24.920	24.753	15	58.704	58.881
24	10.518	10.540	90	25.354	25.216	63	59.852	59.805
105	12.050	12.149	74	25.404	25.460	99	60.910	60.271
20	12.646	12.579	51	25.486	25.561	43	61.078	61.210
54	12.890	12.969	67	27.028	27.027	98	75.738	74.899
93	13.090	13.119	102	28.560	28.676	28	77.674	77.413
70	13.146	13.187	34	29.188	28.840	62	78.468	77.788
81	13.612	13.638	46	31.418	31.388	14	80.352	80.187
80	15.736	15.477	89	32.042	32.000	85	81.304	80.327
92	15.660	15.593	75	30.246	32.211	31	86.892	86.332
36	16.018	16.066	66	32.572	32.562	97	87.878	87.499
53	16.134	16.228	17	33.302	33.260	41	93.988	93.595
35	16.726	16.804	101	35.930	36.073	13	95.124	94.677
104	16.770	16.871	33	38.066	37.960	27	97.944	98.066
19	17.510	17.453	30	66.290	66.410	26	112.398	111.941
45	38.494	38.472	86	66.854	66.538	40	113.208	112.541
65	41.364	39.267	42	75.200	75.153	12	114.780	114.126
79	41.162	41.313	88	41.682	41.715			
39	18.034	18.053	16	44.662	44.522			

3.56 ft.) was the standard weir,* the quantity discharged in each series has been computed by using the heads on this weir, in the formula,

$$Q = m\, h^{1.4927}, \text{ in which log. } m = 0.5239.$$

How well this formula fits the experiments made for rating this weir is shown by the heavy broken line in Fig. 13, Plate XXVII*a*. For a head of 0.151 ft., the computed discharge is 1.01% too small, according to the one experiment with this low head; and, for the highest head used, namely, 0.805 ft., the computed discharge is 0.48% too small. For heads between these limits, the formula gives some discharges a little more than 0.4% too large.

However, as 0.944 ft. is the highest and 0.188 ft. the lowest head used in calculating these discharges, and as neither of these gives a discharge differing more than 0.5% from that actually measured volumetrically; and, further, as the results obtained by Bazin, when ex-

* *Transactions*, Am. Soc. C. E., Vol. XII, p. 110.

PLATE XXIX.
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WEIR MEASUREMENT OF
STREAM FLOW.

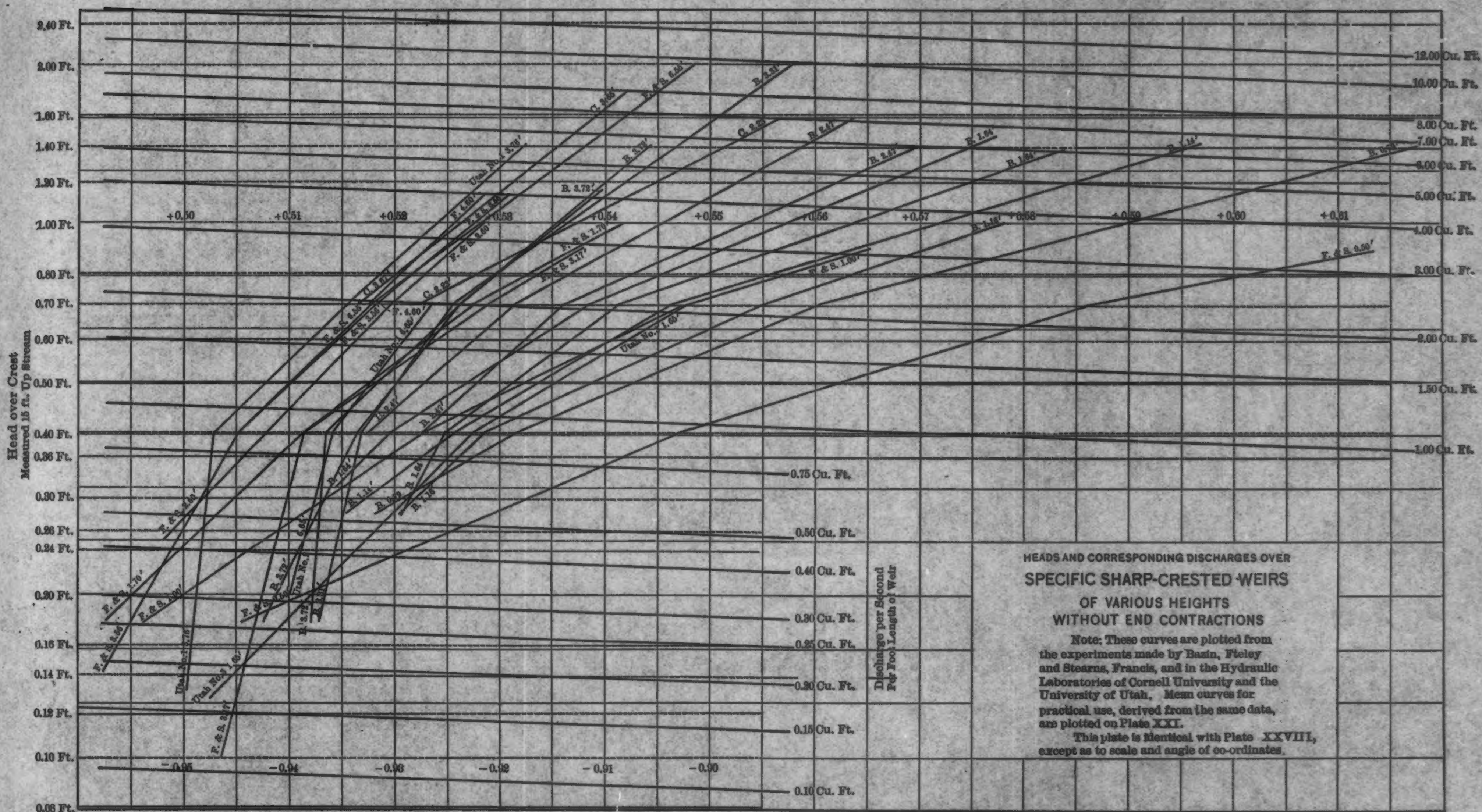


TABLE C.—SHARP-CRESTED WEIRS.

Values of m and n to be used in the equation, $Q = m\,h^n$, for computing the discharge, in cubic feet per second per foot of length of weir, and the limiting value of the heads to be used in the corresponding equation.

(1) No. of table.	(2) Height of weir.	(3) n.	(4) Log. m.	(5) m.	(6) Limits of Head, in Feet. Upper.	(7) Limits of Head, in Feet. Lower.	(8) Name of experimenter.
1	11.25	1.5329	0.5117	3.249	5.00	2.00	Application of Bazin's Formula by Dr. Schoder.
		1.4879	0.5252	3.351	2.00	0.70	
		1.4567	0.5204	3.314	0.70	0.40	
		1.4161	0.5041	3.192	0.40	0.10	
2	11.25	1.5346	0.5116	3.248	4.90	2.00	Application of Bazin's Formula by Professor Williams.
		1.4902	0.5249	3.349	2.00	0.70	
		1.4695	0.5217	3.324	0.70	0.40	
		1.4130	0.4990	3.155	0.40	0.16	
3	11.25	1.5373	0.5099	3.235	3.70	2.00	Cornell Hydraulic Laboratory.
		1.4767	0.5280	3.373	2.00	0.70	
		1.4592	0.5253	3.352	0.70	0.20	
4	6.65	1.5702	0.5087	3.226	4.00	2.00	Cornell Hydraulic Laboratory.
		1.5035	0.5285	3.377	2.00	0.70	
		1.4631	0.5221	3.327	0.70	0.40	
		1.3978	0.4960	3.133	0.40	0.11	
5	6.65	1.5671	0.5101	3.237	5.00	2.00	Application of Bazin's Formula by Professor Williams.
		1.5070	0.5282	3.374	2.00	0.70	
		1.4675	0.5221	3.327	0.70	0.40	
		1.3595	0.4790	3.013	0.40	0.16	
6	6.55	1.4886	0.5294	3.384	0.70	0.40	Weir No. 2, University of Utah Hydraulic Laboratory.
		1.4632	0.5192	3.305	0.40	0.20	
7	6.55	1.5175	0.5282	3.374	2.00	0.70	Fteley and Stearns.
		1.5024	0.5260	3.357	0.70	0.40	
8	4.60	1.5032	0.5257	3.355	1.00	0.70	J. B. Francis.
8A		1.4964	0.5214	3.322	1.00	0.70	J. B. Francis.
9	3.72	1.5123	0.5358	3.434	1.15	0.70	Bazin.
		1.4896	0.5324	3.407	0.70	0.40	
		1.4749	0.5265	3.361	0.40	0.18	
10	3.72	1.5101	0.5356	3.432	1.40	0.70	Bazin.
		1.5024	0.5348	3.422	0.70	0.40	
		1.4537	0.5150	3.273	0.40	0.18	
11	3.72	1.5058	0.5240	3.342	1.40	0.70	Weir No. 1, University of Utah Hydraulic Laboratory.
		1.5019	0.5232	3.336	0.70	0.40	
		1.4549	0.5042	3.193	0.40	0.13	
12	3.65	1.5133	0.5263	3.360	1.75	0.70	Cornell Hyd. Lab.
13	3.56	1.5118	0.5277	3.371	0.90	0.70	Fteley and Stearns.
		1.5029	0.5262	3.351	0.70	0.40	
		1.4784	0.5165	3.285	0.40	0.14	
14	3.31	1.5209	0.5365	3.440	2.00	0.70	Bazin.
		1.4860	0.5311	3.397	0.70	0.40	
		1.4612	0.5212	3.320	0.40	0.18	
15	3.17	1.5348	0.5421	3.484	0.90	0.70	Fteley and Stearns.
		1.5225	0.5402	3.469	0.70	0.40	
		1.4627	0.5162	3.282	0.40	0.10	
16	2.60	1.5075	0.5286	3.378	1.00	0.70	Fteley and Stearns.
		1.4980	0.5271	3.366	0.70	0.40	
		1.4998	0.5279	3.372	0.40	0.25	
17	2.47	1.5621	0.5534	3.495	1.40	0.70	Bazin.
		1.5056	0.5447	3.505	0.70	0.40	
18	2.47	1.5402	0.5456	3.512	1.60	0.70	Bazin.
		1.5074	0.5406	3.472	0.70	0.40	
19	2.23	1.5472	0.5374	3.447	1.60	0.70	Cornell Hyd. Lab.
20	1.70	1.5409	0.5417	3.481	1.00	0.70	Fteley and Stearns.
		1.5169	0.5381	3.452	0.70	0.40	
		1.5026	0.5323	3.406	0.40	0.18	

TABLE C.—(*Continued.*)

(1) No. of table.	(2) Height of weir.	(3) *n*.	(4) Log. *m*.	(5) *m*.	(6) Limits of Head, in Feet. Upper.	(7) Lower.	(8) Name of experimenter.
21	1.64	1.5938	0.5636	3.661	1.40	0.70	Bazin.
		1.5179	0.5519	3.564	0.70	0.40	
		1.4775	0.5357	3.433	0.40	0.28	
22	1.64	1.5716	0.5575	3.610	1.50	0.70	Bazin.
		1.5281	0.5505	3.552	0.70	0.40	
		1.5000	0.5397	3.465	0.40	0.32	
23	1.64	1.5338	0.5608	3.637	0.70	0.40	Weir No. 3, University of Utah Hydraulic Laboratory.
		1.5000	0.5473	3.526	0.40	0.12	
24	1.16	1.5985	0.5780	3.784	1.00	0.70	Bazin.
		1.5595	0.5719	3.732	0.70	0.40	
		1.5025	0.5490	3.459	0.40	0.28	
25	1.14	1.6155	0.5720	3.733	1.40	0.70	Bazin.
		1.5350	0.5595	3.627	0.70	0.40	
		1.5181	0.5526	3.569	0.40	0.28	
26	1.00	1.6142	0.5732	3.743	0.90	0.70	Fteley and Stearns.
		1.5525	0.5635	3.660	0.70	0.40	
		1.5250	0.5524	3.568	0.40	0.18	
27	0.79	1.6389	0.5894	3.885	1.40	0.70	Bazin.
		1.5691	0.5789	3.792	0.70	0.40	
		1.5373	0.5661	3.682	0.40	0.28	
28	0.50	1.7088	0.6265	4.232	0.90	0.70	Fteley and Stearns.
		1.6112	0.6111	4.084	0.70	0.40	
		1.5668	0.5933	3.920	0.40	0.18	

TABLE D.—Broad-Crested Weirs.

Values of m and n to be used in the equation, $Q = m\, h^n$, for computing the discharge, in cubic feet per second per foot of length, and the limiting values of the heads to be used in the corresponding equations.

(1) No. of table.	(2) Width of crest.	(3) *n*.	(4) Log. *m*.	(5) *m*.	(6) Limits of Head, in Feet. Upper.	(7) Lower.	(8) Cornell model.
38	5¾ in.	1.5053	0.5223	3.329	2.00	0.81	XL
		1.7428	0.5445	3.503	0.81	0.36	
		1.4285	0.4055	2.544	0.36	0.12	
39	11⅛ in.	1.5097	0.5207	3.317	2.95	1.72	XLVII
		1.6837	0.4797	3.018	0.70	0.70	
		1.6210	0.4698	2.950	0.40	0.40	
40	19¾ in.	1.5381	0.5076	3.218	4.00	3.00	XLI
		1.6943	0.4402	2.756	3.00	0.74	
		1.4224	0.4072	2.554	0.74	0.11	
41	3.17 ft.	1.6505	0.3925	2.469	3.10	1.50	XLVI
		1.4746	0.4246	2.658	1.50	0.50	
42	5.85 ft.	1.4900	0.4234	2.651	2.00	0.70	XLV
		1.4338	0.4155	2.603	0.70	0.50	
43	8.96 ft.	1.4918	0.4245	2.658	3.10	0.70	XLIV
		1.4306	0.4150	2.600	0.70	0.50	
44	12.25 ft.	1.6006	0.3696	2.342	5.00	3.00	XLII
		1.4590	0.4369	2.735	3.00	1.05	
		1.5237	0.4357	2.727	1.05	0.17	
45	16.29 ft.	1.5305	0.4078	2.557	4.50	2.50	XLIII
		1.4797	0.4276	2.677	2.50	0.70	
46	16.29 ft.	1.5561	0.4301	2.692	1.00	0.76	XLIII*a*
		1.4530	0.4178	2.617	0.76	0.16	

TABLE E.—IRREGULAR-CRESTED WEIRS OF RIGHT-LINE SECTIONS.

Values of m and n to be used in the equation, $Q = m\ h^n$, for computing the discharge, in cubic feet per second per foot of length, and the limiting values of the heads to be used in the corresponding equations.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
				LIMITS OF HEAD, IN FEET.			
No. of table.	n.	Log. m.	m.	Upper.	Lower.	Cornell model.	Dam on weir.
12	1.4978	0.5278	3.371	1.80	0.80	XXI	
47	1.4848	0.5393	3.462	4.00	0.32	XI	
48	1.4479	0.5704	3.719	2.00	0.85	XXII	Cambria.
	1.5912	0.5815	3.815	0.85	0.40		
49	1.5592	0.5570	3.606	2.80	2.00	XXVI	Cambria.
	1.4986	0.5753	3.761	2.00	1.10		
50	1.5472	0.5556	3.594	2.80	1.60	XXIII	Cambria.
	1.5083	0.5633	3.658	1.60	0.40		
51	1.5576	0.5553	3.592	2.80	1.35	XXIV	Cambria.
52	1.5200	0.5640	3.664	1.40	0.32	XXV	Cambria.
53	1.6705	0.4685	2.941	4.00	1.90	XXVII	Lawrence.
	1.5180	0.4972	3.142	1.90	0.32		
54	1.5278	0.5422	3.485	1.00	0.40	XX	
55	1.6311	0.4625	2.901	3.65	1.60	XXVII	Lawrence.
	1.4920	0.4901	3.091	1.60	0.63		
	1.4507	0.4819	3.033	0.63	0.28		
65	1.5365	0.5550	3.589	4.00	1.00	L	
	1.4558	0.5550	3.589	1.00	0.35		
66	1.5343	0.5563	3.600	4.00	1.00	LI	
	1.6363	0.5563	3.600	1.00	0.28		

perimenting on the same weirs at different times, differ by percentages much greater than this, it is not deemed necessary to repeat this work, using the formulas adopted for this weir instead of that above given.

As a matter of fact, the discharges for all the runs over the Fteley and Stearns weirs had been computed by the formula, $q = m\ h^{1.4927}$, in which log. $m = 0.5239$, or $m = 3.341$, and these results were thought to be very satisfactory until the present method of determining equations had been discovered.

The heads on the Fteley and Stearns weir of height 3.17 ft., and having a length of 5.00 ft.,* were measured with a piezometer which took the water pressure from near the bottom of the canal 6 ft. up stream from the weir.†

The Cornell float piezometer was designed to duplicate this device used for measuring heads by Fteley and Stearns; therefore, the cor-

* *Transactions*, Am. Soc. C. E., Vol. XII, p. 56.

† *Transactions*, Am. Soc. C. E., Vol. XII, p. 53.

TABLE F.—Irregular-Crested Weirs of Right-line and Curved Sections.

Values of m and n to be used in the equation, $Q = m\,h^n$, for computing the discharge, in cubic feet per second per foot of length, and the limiting values of the heads to be used in the corresponding equations.

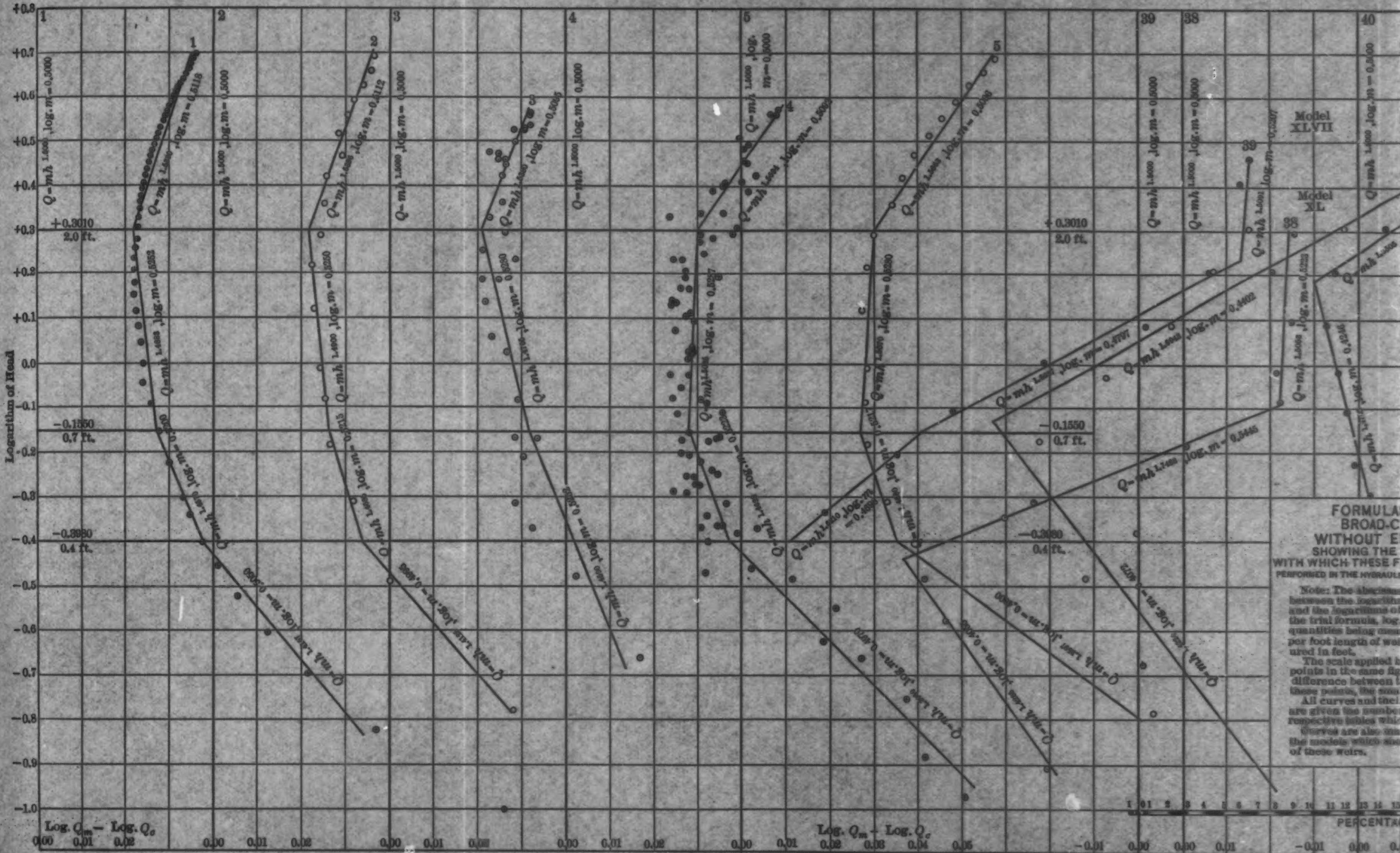
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
No. of table.	n	Log. m.	m.	Limits of Head, in Feet. Upper.	Lower.	Cornell model.	Dam on weir
56	1.5217	0.4790	3.013	1.40	0.56	Francis model.	Lawrence.
57	1.5322	0.5310	3.396	3.20	0.40	XXX	Plattsburg.
58	1.5716	0.5451	3.508	2.80	0.56	XXXIII	Plattsburg.
59	1.6226	0.5398	3.466	3.20	0.63	XXXIV	Plattsburg.
60	1.6063	0.5163	3.283	4.00	0.48	XXXV	Chambly.
	1.4576	0.4846	3.052	0.48	0.08		
61	1.5941	0.5217	3.324	2.80	1.25	XXXVI	Chambly.
	1.5555	0.5255	3.354	1.25	0.48		
62	1.4074	0.5736	3.746	3.55	1.60	XXXVIII	Dolgeville.
	1.5752	0.5401	3.468	1.60	0.40		
63	1.4425	0.5546	3.586	4.00	1.25	XXXIX	Dolgeville.
	1.5081	0.5481	3.533	1.25	0.40		
64	1.5220	0.5570	3.606	3.20	1.25	XLVIII	
	1.4818	0.5609	3.638	1.25	0.50		
65	1.5365	0.5550	3.589	4.00	1.00	L	
	1.4558	0.5550	3.589	1.00	0.35		
67	1.5180	0.5582	3.616	3.30	0.63	XLIX	

rections necessary to reduce heads thus measured to equivalent 15-ft. tape readings, as shown on Plate XXXIV, have been applied, except that for heads of less than 1 ft. the correction has been made to decrease uniformly from 0.007 for a 1-ft. head to 0 at the origin.

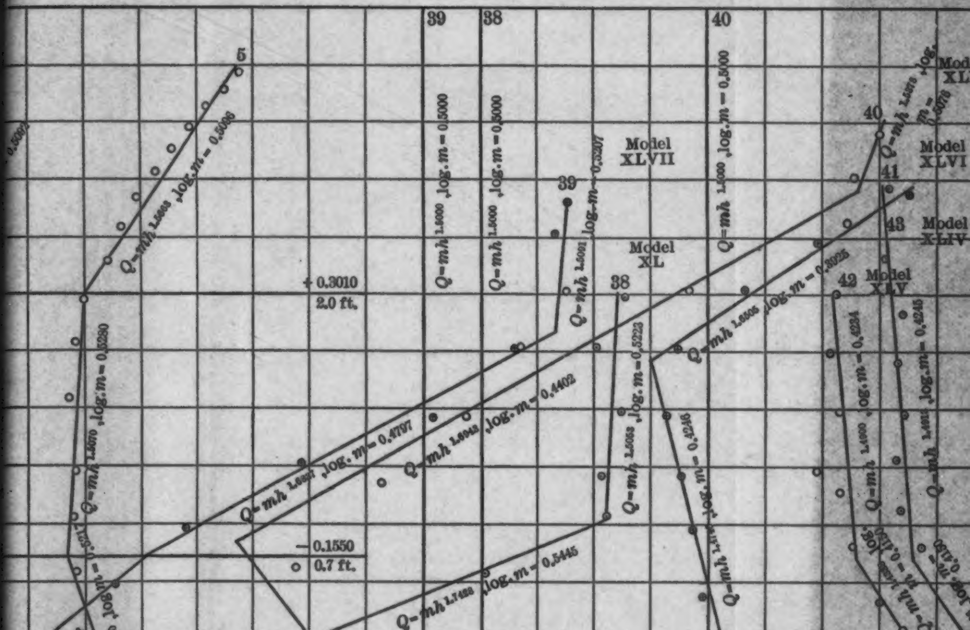
(*d*).—*Bazin Experiments.*—The results of Bazin's experiments,* Series 1 to 10, inclusive, and of computations made on them, are given in Tables 9, 10, 14, 17, 18, 21, 22, 24, 25, and 27, and the plottings showing the trial equation, the adopted equations, the results of the final diagram, and the degree of accuracy with which these fit the original experiments, are shown in Figs. 9, 10, 14, of Plate XXVII*a*, and Figs. 17, 18, 21, 22, 24, 25, and 27 of Plate XXVII*b*.

The curves of these equations are brought together on Plates XXVIII and XXIX, where they, with others of this class, are plotted on the same axes, for purposes of comparison.

* Experiments by M. Bazin, Inspector-General of Bridges and Highways, published in *Annales des Ponts et Chaussées*, October, 1888, and January, 1890. Translated by Arthur Marichal and John C. Trautwine, Jr., and published in Volumes VII, IX, and X, *Proceedings*, Engineers' Club of Philadelphia.



Logarithm of Head
+0.3010
2.0 ft.
−0.1550
0.7 ft.
−0.3980
0.4 ft.
Log. Q_m − Log. Q_c
Model XLVII
Model XL
FORMULAS
BROAD-C
WITHOUT E
SHOWING THE
WITH WHICH THESE F
PERCENTAG



FORMULAS ADOPTED FOR
BROAD-CRESTED WEIRS
WITHOUT END CONTRACTIONS
SHOWING THE DEGREE OF ACCURACY
WITH WHICH THESE FIT THE ACTUAL EXPERIMEN
PERFORMED IN THE HYDRAULIC LABORATORY OF CORNELL UNIVERS

Note: The abscissas represent the differences between the logarithms of the quantities measured and the logarithms of the quantities computed by the trial formula, $\log. Q_c = 0.5000 + 1.5000 \log. h$, the quantities being measured in cubic feet per second per foot length of weir and the heads being measured in feet.

The scale applied horizontally between any two points in the same figure gives the percentage of difference between the discharges represented by these points, the smaller discharge being the base.

All curves and their respective reference lines are given the numbers used to designate the respective tables which contain the original data.

Curves are also marked with the numbers of the models which show the actual cross-sections of these weirs.

PLATE XXX.
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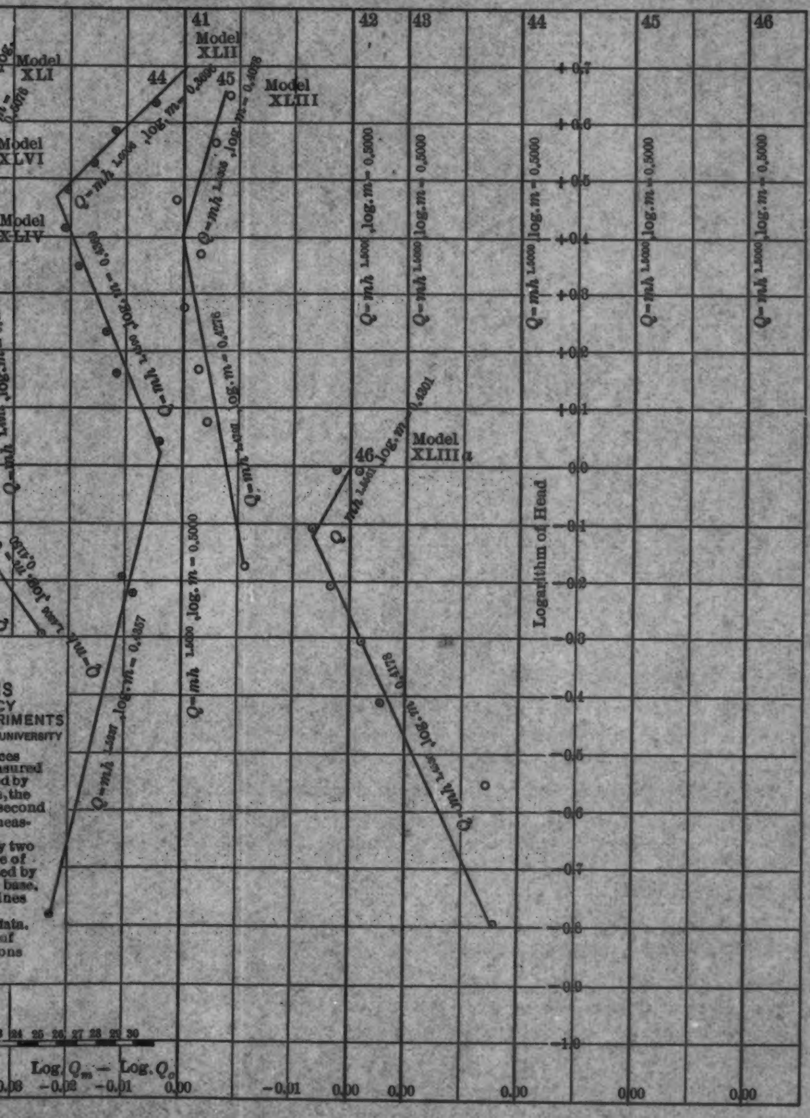


The method used by Bazin for measuring heads is described on pages 1271 and 1272. The heads, as originally measured, have been reduced to equivalent 15-ft. tape readings by applying to them the necessary corrections, as shown on Plate XXXIV. The heads, as originally measured, and the equivalent tape readings, are given in the above-named tables.

The quantities of discharge, for Bazin's Series 1, 2, and 3, are taken from the tabulation of his experiments, but he does not give the quantities of discharge for the other seven series; therefore, for the purpose of determining these quantities, the curve shown on Plate XXXV was constructed by using the heads, in meters, as ordinates and the quantities of discharge, in cubic feet per second per foot of length of weir, as abscissas.

The points plotted were determined by using the heads and quantities given in Bazin's Series 1 and 2. These two series contain all the experiments made on Bazin's standard weir.

The quantities of discharge for Series 1 and 2 were measured volumetrically, as were also those for Series 3. For the remaining seven series, however, only the head on the standard weir is given; and with this quantity, given in meters, by using the curve on Plate XXXV, the discharges, in cubic feet per second per foot of length of weir, were found.

(e).—Cornell Experiments on Sharp-Crested Weirs.—The Cornell experiments of this class, including Series XXIX, XXIX*a*, XI, and XXI (Figs. 16, 17, 18, and 19), and the results of computations made on them, are given in Tables 3, 4, 12, and 19, and in Figs. 3, 4, and 12, of Plate XXVII*a*, and Fig. 19 of Plate XXVII*b*.

The curves of these equations are brought together on Plates XXVIII and XXIX, where they, with others of this class, are plotted on the same axes, for purposes of comparison.

The heads in these experiments were measured with the tapes named in the tables. The experiments in Series XXIX and XXIX*a*, marked *s*, were made in April, 1903; those marked *f* were made in October, 1903. The experiments in Series XI were made in October, 1902; those in Series XXI, in November, 1902.

(f).—Utah Experiments.—The Utah experiments of this class were made on three weirs: No. 1, 3.72 ft. high and 1.77 ft. long; No. 2, 6.65 ft. high and 6.55 ft. long; and No. 3, 1.64 ft. high and 6.53 ft. long

TABLE F.—Irregular-Crested Weirs of Right-line and Curved Sections.

Values of m and n to be used in the equation, $Q = m\ h^n$, for computing the discharge, in cubic feet per second per foot of length, and the limiting values of the heads to be used in the corresponding equations.

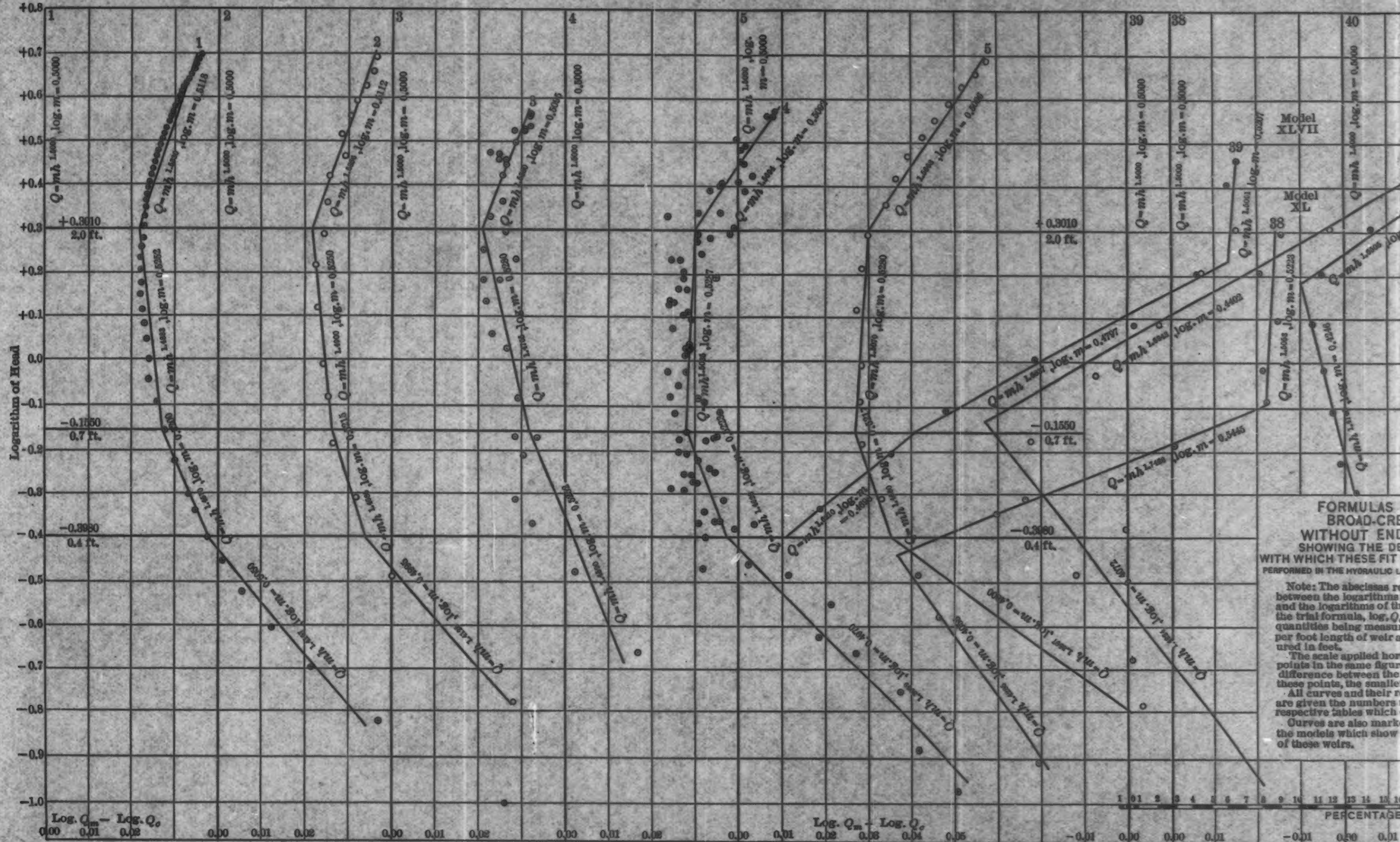
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
No. of table.	n	Log. m.	m.	Limits of Head, in Feet. Upper.	Lower.	Cornell model.	Dam on weir
56	1.5217	0.4790	3.013	1.40	0.56	Francis model.	Lawrence.
57	1.5322	0.5310	3.396	3.20	0.40	XXX	Plattsburg.
58	1.5716	0.5451	3.508	2.80	0.56	XXXIII	Plattsburg.
59	1.6226	0.5398	3.466	3.20	0.63	XXXIV	Plattsburg.
60	1.6063	0.5163	3.283	4.00	0.48	XXXV	Chambly.
	1.4576	0.4846	3.052	0.48	0.08		
61	1.5941	0.5217	3.324	2.80	1.25	XXXVI	Chambly.
	1.5555	0.5255	3.354	1.25	0.48		
62	1.4074	0.5736	3.746	3.55	1.60	XXXVIII	Dolgeville.
	1.5752	0.5401	3.468	1.60	0.40		
63	1.4425	0.5546	3.586	4.00	1.25	XXXIX	Dolgeville.
	1.5081	0.5481	3.533	1.25	0.40		
64	1.5220	0.5570	3.606	3.20	1.25	XLVIII	
	1.4818	0.5609	3.638	1.25	0.50		
65	1.5365	0.5550	3.589	4.00	1.00	L	
	1.4558	0.5550	3.589	1.00	0.35		
67	1.5180	0.5582	3.616	3.30	0.63	XLIX	

rections necessary to reduce heads thus measured to equivalent 15-ft. tape readings, as shown on Plate XXXIV, have been applied, except that for heads of less than 1 ft. the correction has been made to decrease uniformly from 0.007 for a 1-ft. head to 0 at the origin.

(*d*).—*Bazin Experiments.*—The results of Bazin's experiments,* Series 1 to 10, inclusive, and of computations made on them, are given in Tables 9, 10, 14, 17, 18, 21, 22, 24, 25, and 27, and the plottings showing the trial equation, the adopted equations, the results of the final diagram, and the degree of accuracy with which these fit the original experiments, are shown in Figs. 9, 10, 14, of Plate XXVII*a*, and Figs. 17, 18, 21, 22, 24, 25, and 27 of Plate XXVII*b*.

The curves of these equations are brought together on Plates XXVIII and XXIX, where they, with others of this class, are plotted on the same axes, for purposes of comparison.

* Experiments by M. Bazin, Inspector-General of Bridges and Highways, published in *Annales des Ponts et Chaussées*, October, 1888, and January, 1890. Translated by Arthur Marichal and John C. Trautwine, Jr., and published in Volumes VII, IX, and X, *Proceedings*, Engineers' Club of Philadelphia.



Logarithm of Head
+0.3010
2.0 ft.
-0.1550
0.7 ft.
-0.3980
0.4 ft.
Log. Q_m − Log. Q_c
Model XLVII
Model XL
FORMULAS
BROAD-CRE
WITHOUT END
SHOWING THE DE
WITH WHICH THESE FIT
PERFORMED IN THE HYDRAULIC L
Note: The abscissas re
between the logarithms
and the logarithms of the
the trial formula, log. Q_c
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ured in feet.
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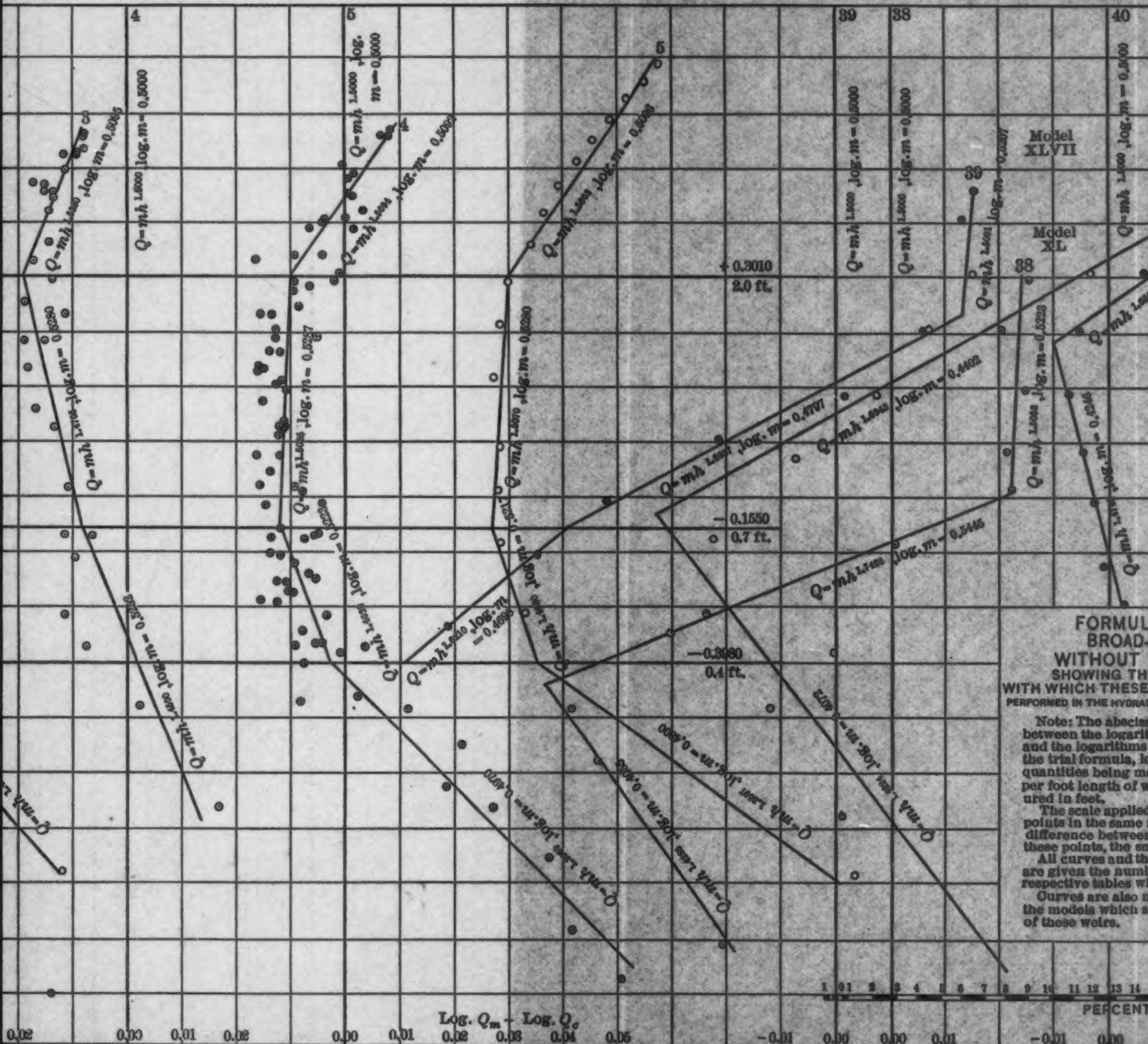
PERCENTAGE



Model XLVII
Model XL
+0.3010
2.0 ft.
—0.1550
0.7 ft.
—0.3980
0.4 ft.
Log. Q_m — Log. Q_c
PERCENT
FORMUL
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Note: The abscis
between the logarit
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Curves are also
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PLATE XXX.
TRANS. AM. SOC. CIV. ENGRS.
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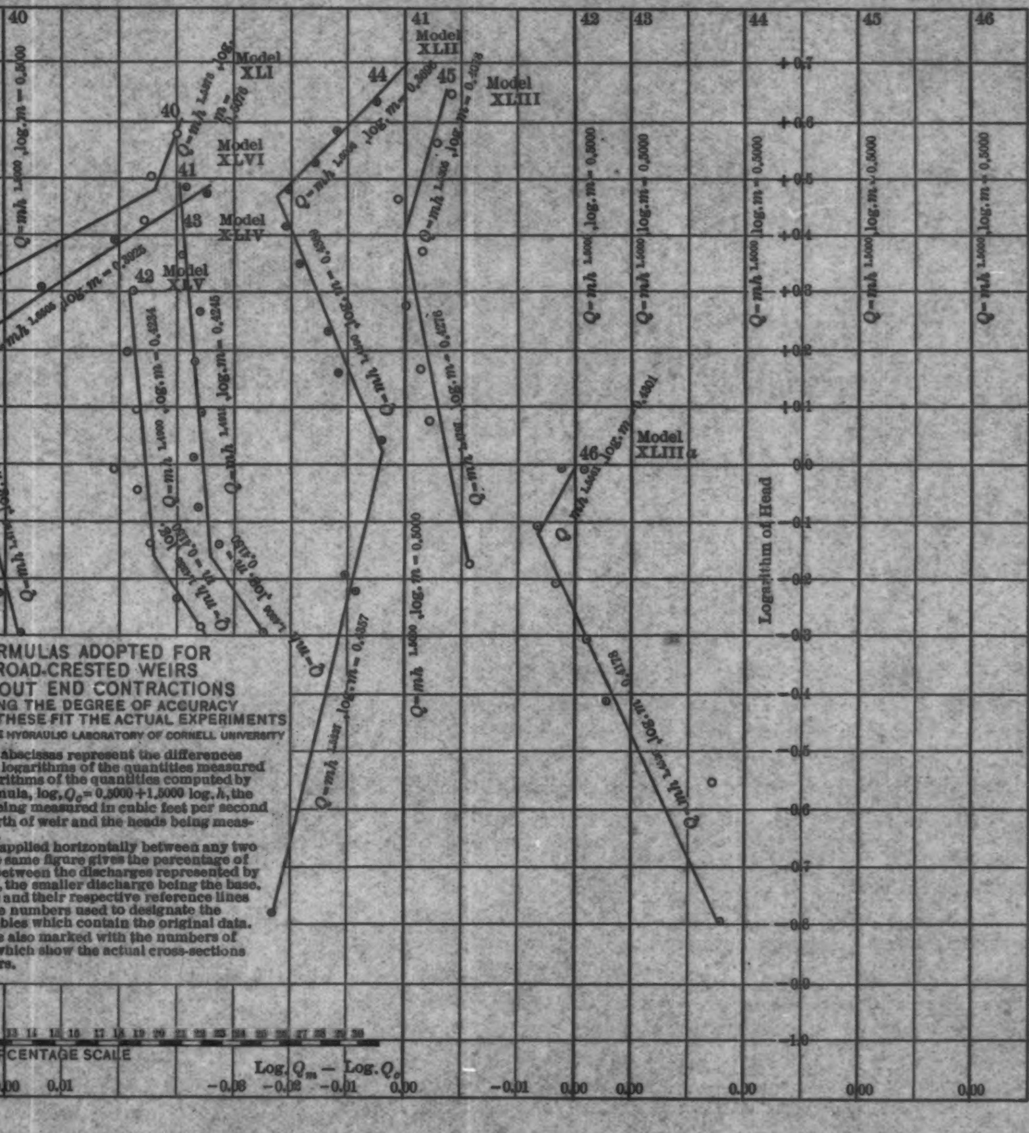


The method used by Bazin for measuring heads is described on pages 1271 and 1272. The heads, as originally measured, have been reduced to equivalent 15-ft. tape readings by applying to them the necessary corrections, as shown on Plate XXXIV. The heads, as originally measured, and the equivalent tape readings, are given in the above-named tables.

The quantities of discharge, for Bazin's Series 1, 2, and 3, are taken from the tabulation of his experiments, but he does not give the quantities of discharge for the other seven series; therefore, for the purpose of determining these quantities, the curve shown on Plate XXXV was constructed by using the heads, in meters, as ordinates and the quantities of discharge, in cubic feet per second per foot of length of weir, as abscissas.

The points plotted were determined by using the heads and quantities given in Bazin's Series 1 and 2. These two series contain all the experiments made on Bazin's standard weir.

The quantities of discharge for Series 1 and 2 were measured volumetrically, as were also those for Series 3. For the remaining seven series, however, only the head on the standard weir is given; and with this quantity, given in meters, by using the curve on Plate XXXV, the discharges, in cubic feet per second per foot of length of weir, were found.

(e).—Cornell Experiments on Sharp-Crested Weirs.—The Cornell experiments of this class, including Series XXIX, XXIX*a*, XI, and XXI (Figs. 16, 17, 18, and 19), and the results of computations made on them, are given in Tables 3, 4, 12, and 19, and in Figs. 3, 4, and 12, of Plate XXVII*a*, and Fig. 19 of Plate XXVII*b*.

The curves of these equations are brought together on Plates XXVIII and XXIX, where they, with others of this class, are plotted on the same axes, for purposes of comparison.

The heads in these experiments were measured with the tapes named in the tables. The experiments in Series XXIX and XXIX*a*, marked *s*, were made in April, 1903; those marked *f* were made in October, 1903. The experiments in Series XI were made in October, 1902; those in Series XXI, in November, 1902.

(f).—Utah Experiments.—The Utah experiments of this class were made on three weirs: No. 1, 3.72 ft. high and 1.77 ft. long; No. 2, 6.65 ft. high and 6.55 ft. long; and No. 3, 1.64 ft. high and 6.53 ft. long

(see Figs 7, 8, 9, and 13). The water was caught and measured in the concrete measuring basin of the hydraulic laboratory of the University of Utah (Figs. 7, 8, 11, and 13). The heads were measured with the tapes named in the tables. These experiments were made by the students doing work in this laboratory from October, 1907, to May, 1911. The results of the experiments and of computations made on them are shown in Tables 6, 11, and 23, and in Figs. 6 and 11, of Plate XXVII*a*, and Fig. 23 of Plate XXVII*b*.

The curves of these equations are brought together on Plates XXVIII and XXIX, where they, with others of this class, are plotted on the same axes, for purposes of comparison.

(g).—Bazin Formula and Cornell Standard Weirs.—Besides the curves for the foregoing experiments on sharp-crested weirs without end contractions, there are also shown on Plate XXVII*a* three curves which give the results obtained by substituting assumed heads in Bazin's formula (see page 1205) for weirs of this class.

Table 1 gives the results E. W. Schoder, Assoc. M. Am. Soc. C. E., obtained by substituting assumed heads in Bazin's formula for a weir of height 11.25 ft., also some computations on these results. Fig. 1, Plate XXVII*a*, gives the trial equation and the adopted equations applied to these results, and shows the degree of accuracy with which they fit.

Tables 2* and 5 give the results Professor Williams obtained by substituting assumed heads in Bazin's formula for weirs of heights 11.25 ft. and 6.65 ft., and some computations on these results.

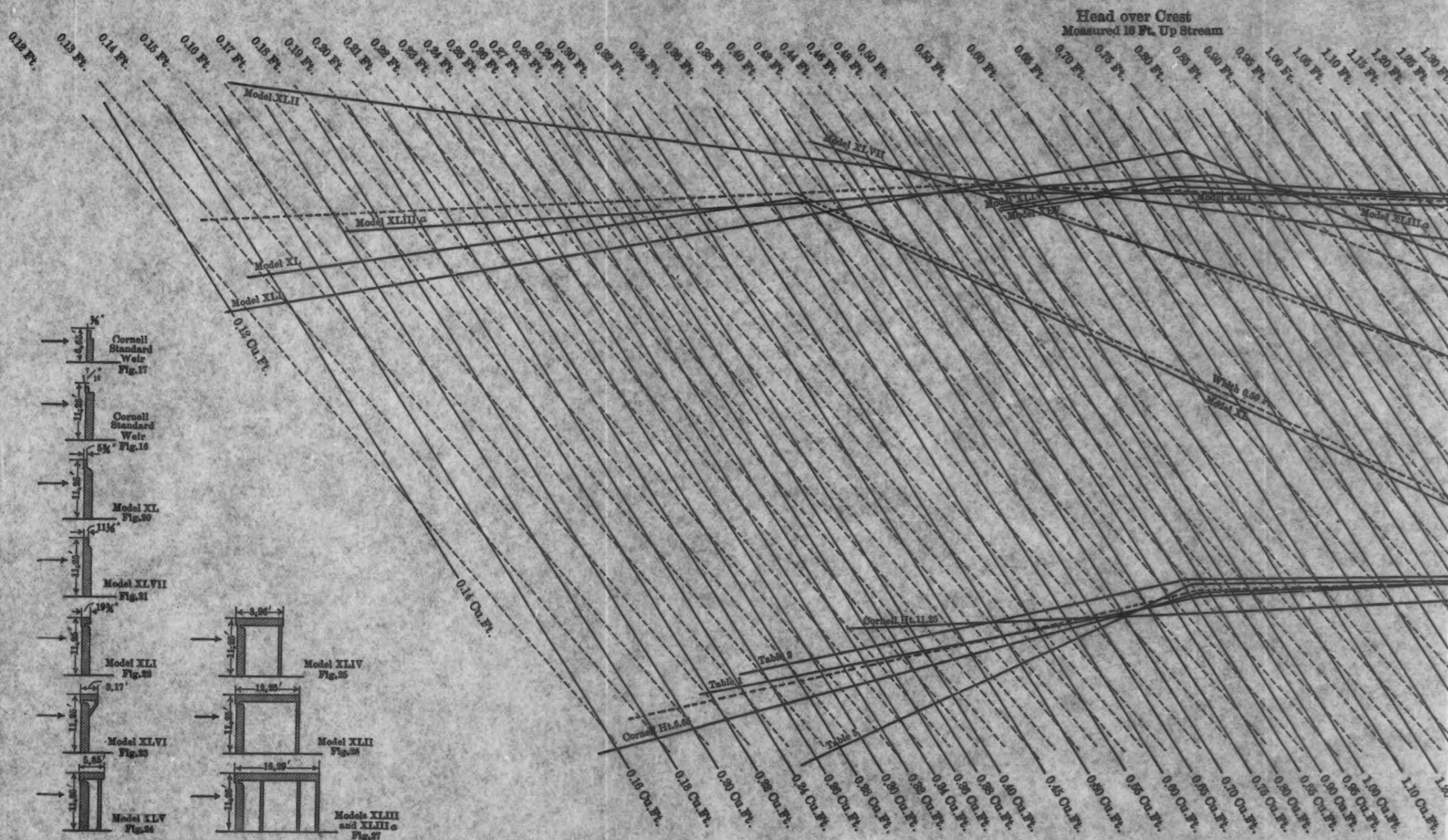
Figs. 2 and 5, Plate XXVII*a*, give the trial equation and the adopted equations applied to these results.

These last two curves are of especial interest. The data which determined them, with the heads, as originally assumed, are the same as those used for constructing the diagrams (page 1273) in the Hydraulic Laboratory of Cornell University; from these diagrams the discharges of these two standard weirs were read.

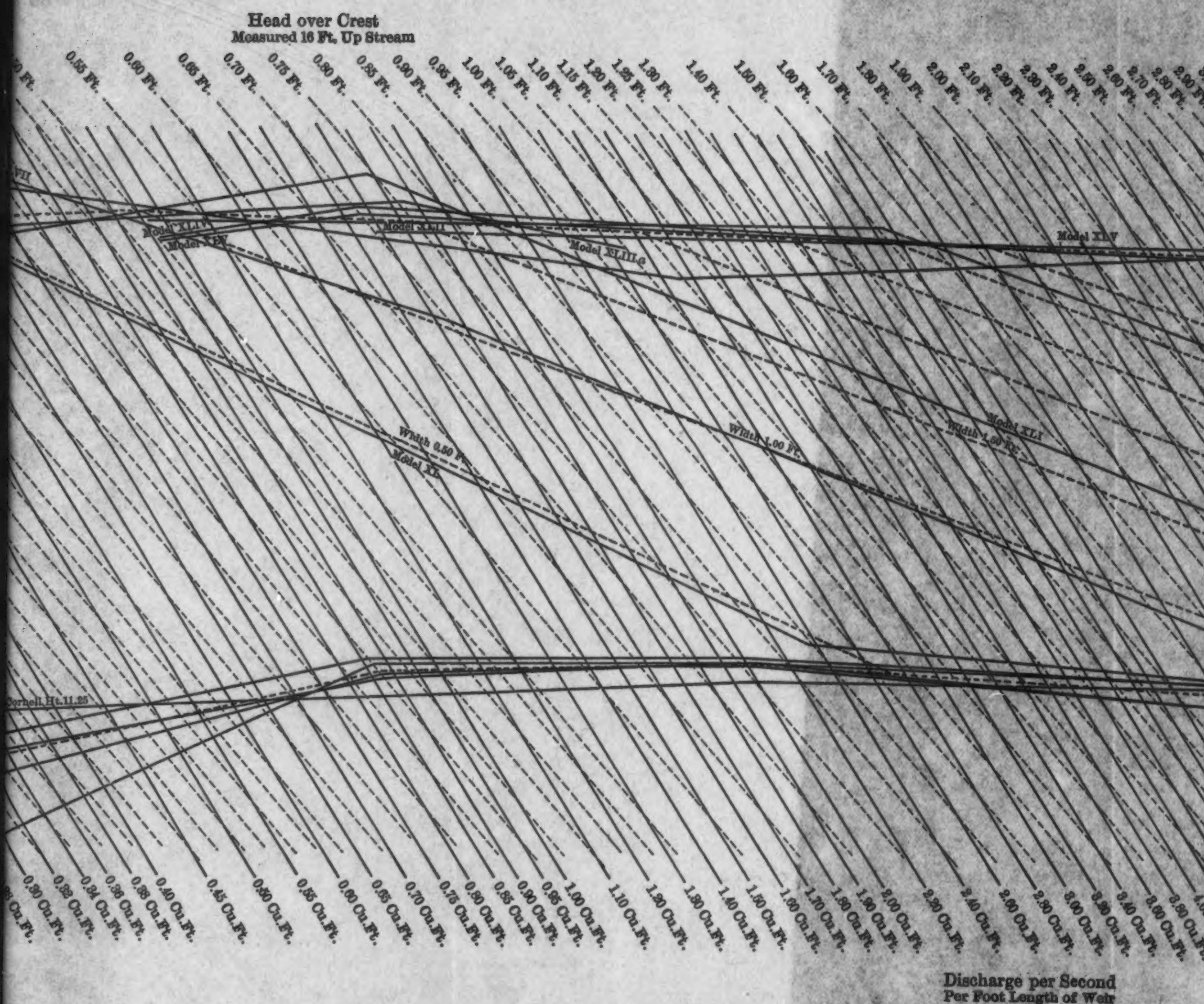
(2).—For Broad-Crested Weirs.

The broad-crested weirs herein considered were built over the weir at the upper end of the canal in the Hydraulic Laboratory of Cornell University. General cross-sections of these models are shown on Plate

* Tables 2 to 67, with the exception of Tables 6, 11, and 23, are not reproduced herein, but are filed in the Library of the Society.



Head over Crest
Measured 16 Ft. Up Stream
Model XLII
Model XLVII
Model XLIII a
Model XL
Model XLI
Model XLIV
Model XLV
Width 0.50 Ft.
Cornell Ht. 11.25'
Cornell Ht. 6.55'
Table 5
0.12 Cu. Ft.
0.14 Cu. Ft.
Cornell Standard Weir Fig. 17
Cornell Standard Weir Fig. 16
Model XL Fig. 20
Model XLVII Fig. 21
Model XLI Fig. 22
Model XLVI Fig. 23
Model XLV Fig. 24
Model XLIV Fig. 25
Model XLII Fig. 26
Models XLIII and XLIII a Fig. 27


Head over Crest
Measured 16 Ft. Up Stream
Model XLIV
Model XLV
Model XLIII
Model XLIII C
Model XLV
Width 0.50 Ft.
Model XL
Width 1.00 Ft.
Width 1.50 Ft.
Model XLI
Cornell Ht. 11.25
Discharge per Second
Per Foot Length of Weir

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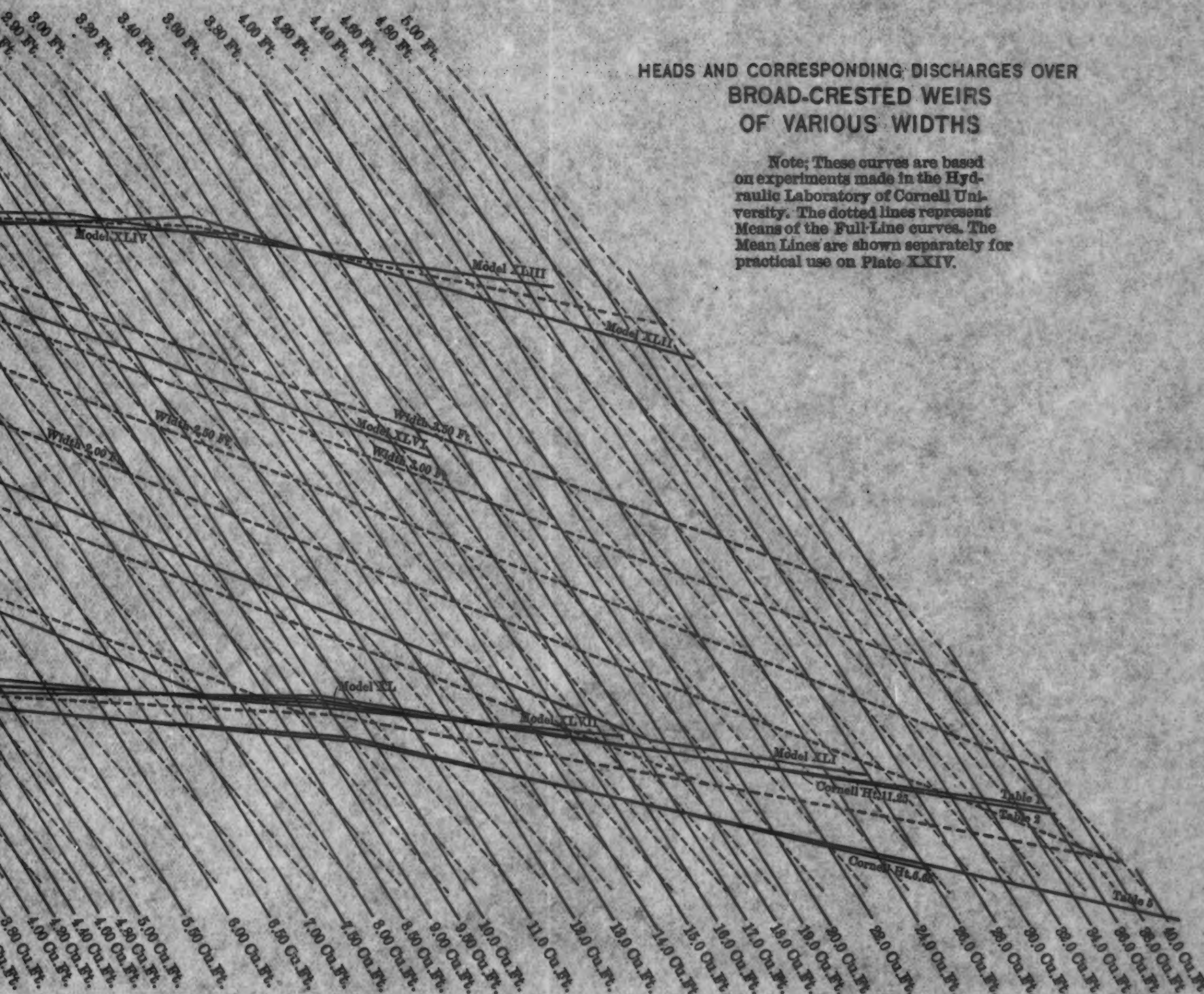
HEADS AND CORRESPONDING DISCHARGES OVER BROAD-CRESTED WEIRS OF VARIOUS WIDTHS

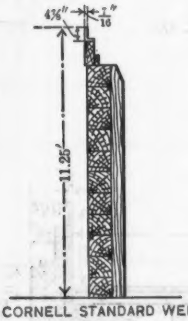
Note: These curves are based on experiments made in the Hydraulic Laboratory of Cornell University. The dotted lines represent Means of the Full-Line curves. The Mean Lines are shown separately for practical use on Plate XXIV.



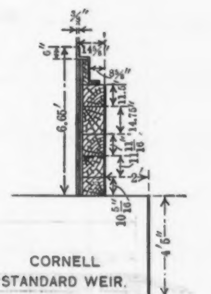
XXXI, and detailed drawings are reproduced in Figs. 16, 17, and 20 to 27, inclusive. The water after passing over these models flowed through the canal and was measured by the lower standard weir, of height 6.65 ft. The space under the falling sheet in these experiments was only partly aerated. The experiments were made during June and July, 1903.

The results of these experiments are given in Tables 1 to 5 and 38 to 46, inclusive, and the plottings showing the trial equation, the adopted equations, the results of the final diagram, and the degree of accuracy attained, are shown on Plate XXX. The curves of these equations are brought together on Plate XXXI, where they are plotted on the same axes, for the purpose of comparing the results.

The adopted equations for these weirs and the limits of the heads between which each may be applied are given in Table D.



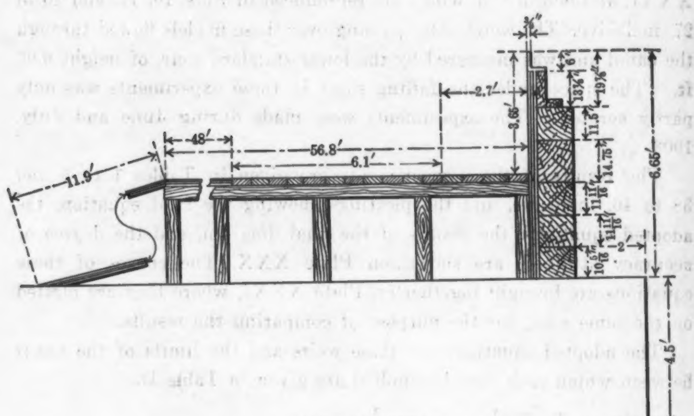
CORNELL STANDARD WEIR.
SERIES XXIX.
FIG. 16.



CORNELL
STANDARD WEIR.
SERIES XXIXa.
FIG. 17.

(3).—For Irregular Weirs with Cross-Sections of Right Lines.

All the models of weirs of this class were constructed in the Cornell Hydraulic Laboratory, at the lower end of the canal, over the weir of height 6.65 ft., and the water flowing over them was measured by the upper weir, of height 11.25 ft. General cross-sections of these models are shown on Plate XXV, and detailed drawings are reproduced in Figs. 17 to 19, 28 to 34, inclusive, 43 and 44. The results of these experiments are given in Tables 3, 4, 12, 37 to 55, inclusive, and also 65 and 66. The plottings giving both the trial equation and the adopted equations and showing their accuracy are shown on Plates XXXII*a* and XXXII*b*. The curves of these equations are brought to-

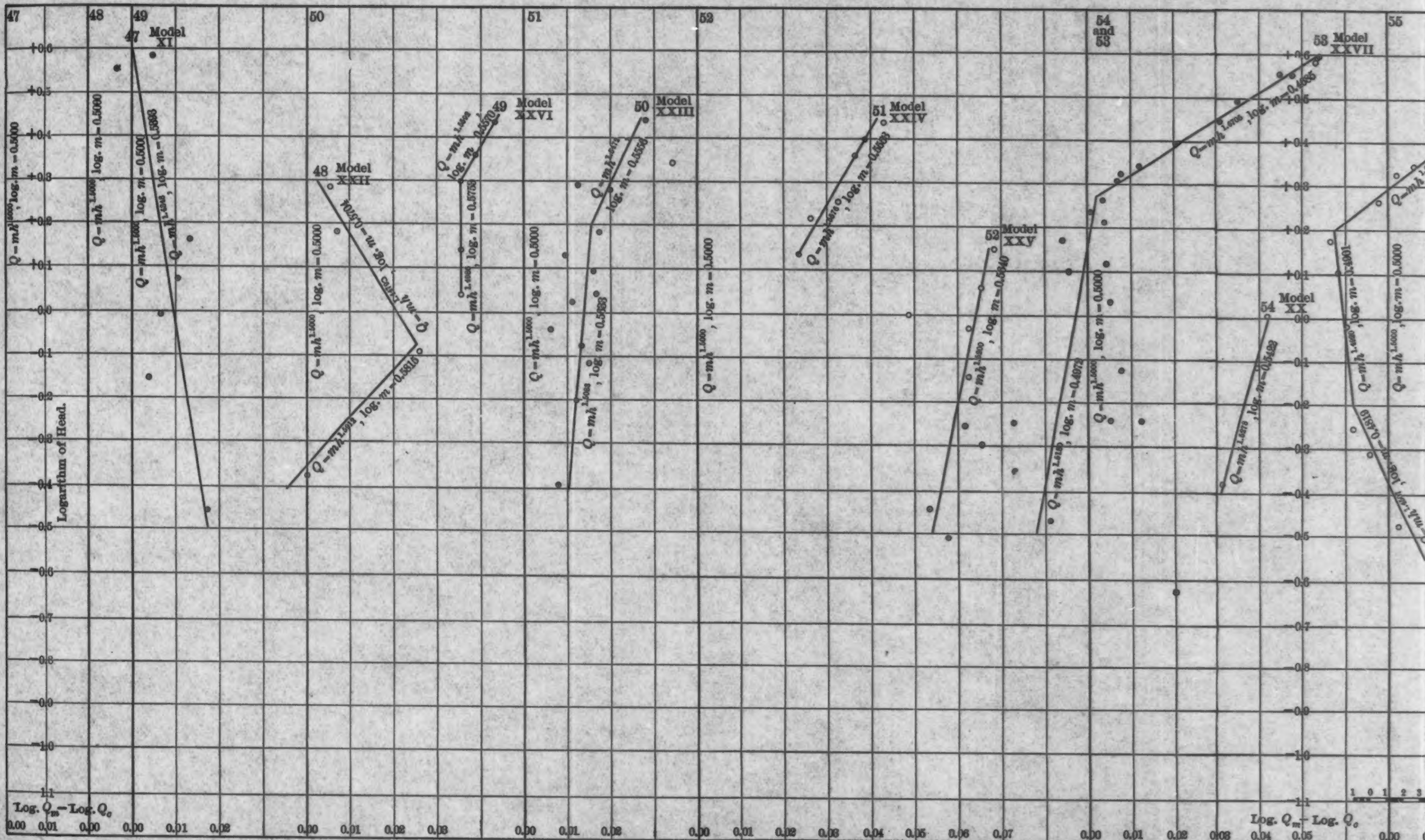


CORNELL MODEL XXI.

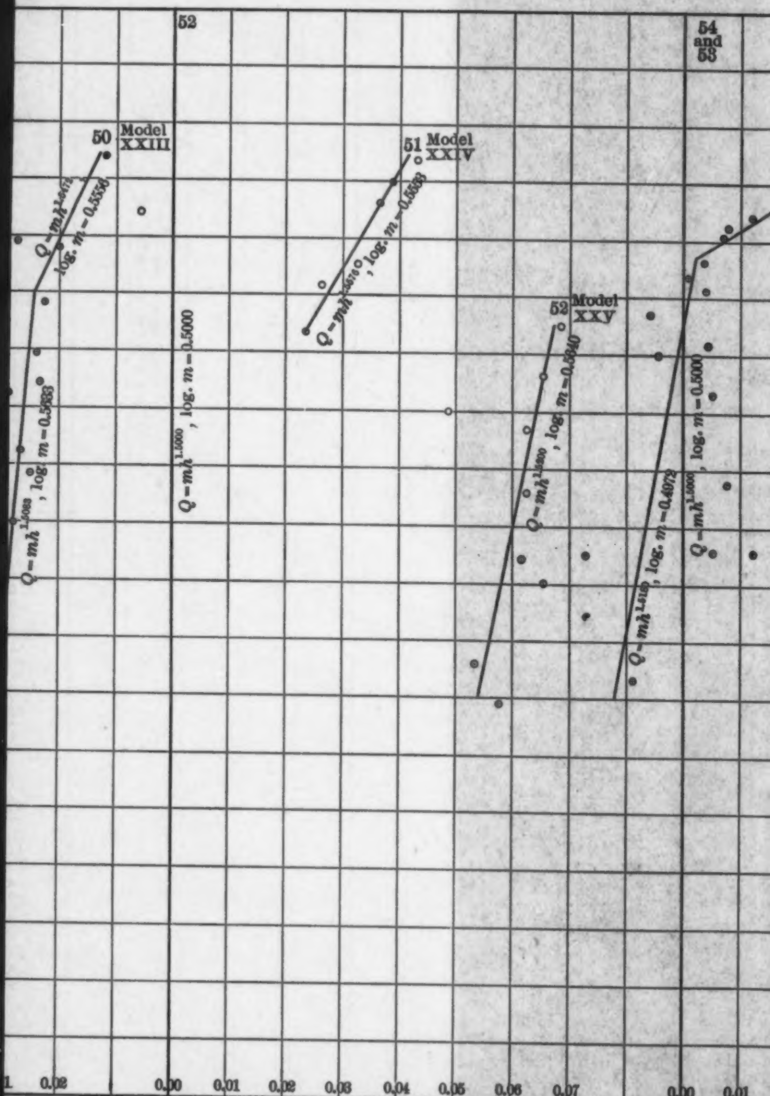
FIG. 18.

CORNELL MODEL XI.

FIG. 19.



47
48
49
50
51
52
54 and 53
55
47 Model XI
48 Model XXII
49 Model XXVI
50 Model XXIII
51 Model XXIV
52 Model XXV
54 Model XX
53 Model XXVII
Logarithm of Head.
Log. Qm − Log. Qo



50 Model XXIII
$Q=mh^{1.5083}$, log. $m=0.5583$
log. $m=0.5556$
52
$Q=mh^{1.5000}$, log. $m=0.5000$
51 Model XXIV
log. $m=0.5583$
52 Model XXV
$Q=mh^{1.5200}$, log. $m=0.5840$
54 and 53
$Q=mh^{1.5150}$, log. $m=0.4972$
$Q=mh^{1.5000}$, log. $m=0.5000$
0.02
0.00
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.00
0.01

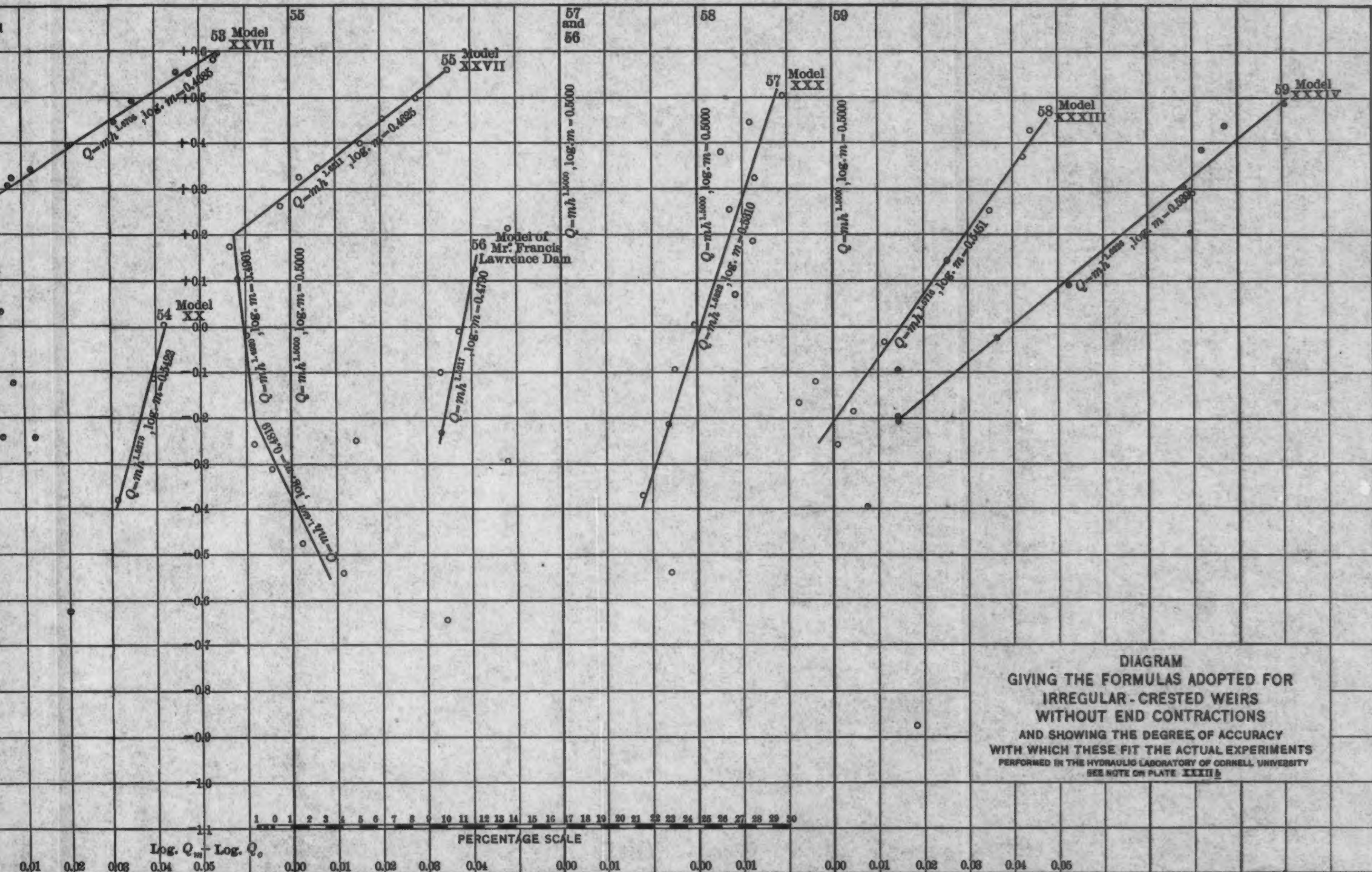
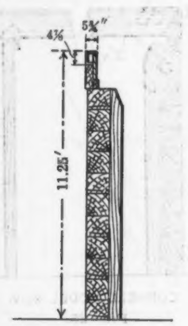


DIAGRAM
GIVING THE FORMULAS ADOPTED FOR
IRREGULAR-CRESTED WEIRS
WITHOUT END CONTRACTIONS
AND SHOWING THE DEGREE OF ACCURACY
WITH WHICH THESE FIT THE ACTUAL EXPERIMENTS
PERFORMED IN THE HYDRAULIC LABORATORY OF CORNELL UNIVERSITY
SEE NOTE ON PLATE XXXIIb

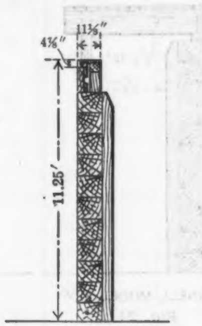
gether, with others of a similar character, on Plate XXV, where, for purposes of comparison, they are plotted on the same axes. The experiments of Series XX and also those of Series XXII, XXIII, XXIV, and XXVI were made in November, 1902. Three of the runs or experiments of Series XXVII were made in December, 1902, the others in April, 1903.

In Series XXIV, XXV, and XXVI there was no aeration behind the falling sheet, but in the other series a free access of air was provided.

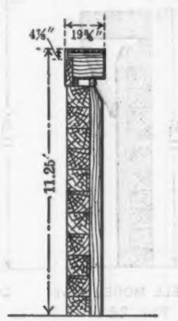
The adopted equations for these weirs and the limits of the heads between which they may be applied are given in Table E.



CORNELL MODEL XL.
FIG. 20.



CORNELL MODEL XLVII.
FIG. 21.



CORNELL MODEL XLI.
FIG. 22.

(4).—For Irregular Weirs with Cross-Sections of Right Lines and Curves.

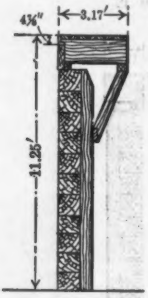
Most of the models of the weirs and dams of this class were built in the Cornell Laboratory over the high weir at the upper end of the canal. General cross-sections of these models are shown on Plate XXVI, and detailed drawings are reproduced in Figs. 16, 35 to 43, inclusive, and 45. Provision was made in all cases for a free access of air behind the falling sheet. These experiments were made during May and June, 1903.

The results obtained are given in Tables 56 to 65, inclusive, and in Table 67, and the plottings, with both the trial equation and the adopted equations, also the degree of accuracy attained, are shown on Plates XXXII*a* and XXXII*b*.

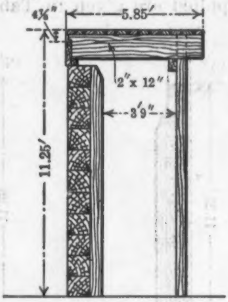
The curves of these equations are brought together on Plate XXVI. Table F gives the adopted equations for these weirs and the limits of the heads between which each may be applied.

G.—Similarity of the Curves That Represent the Adopted Formulas.

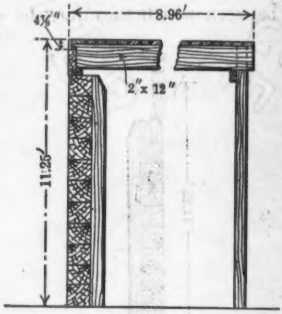
Before taking up a comparison of the discharges over weirs of similar or nearly similar cross-sections, attention will be drawn to Plates XXVII*a*, XXVII*b*, XXX, XXXII*a*, and XXXII*b*, on which the adopted equations and their loci for sharp crests, broad crests, and irregular crests, respectively, are shown.



CORNELL MODEL XLVI.
FIG. 23.



CORNELL MODEL XLV.
FIG. 24.

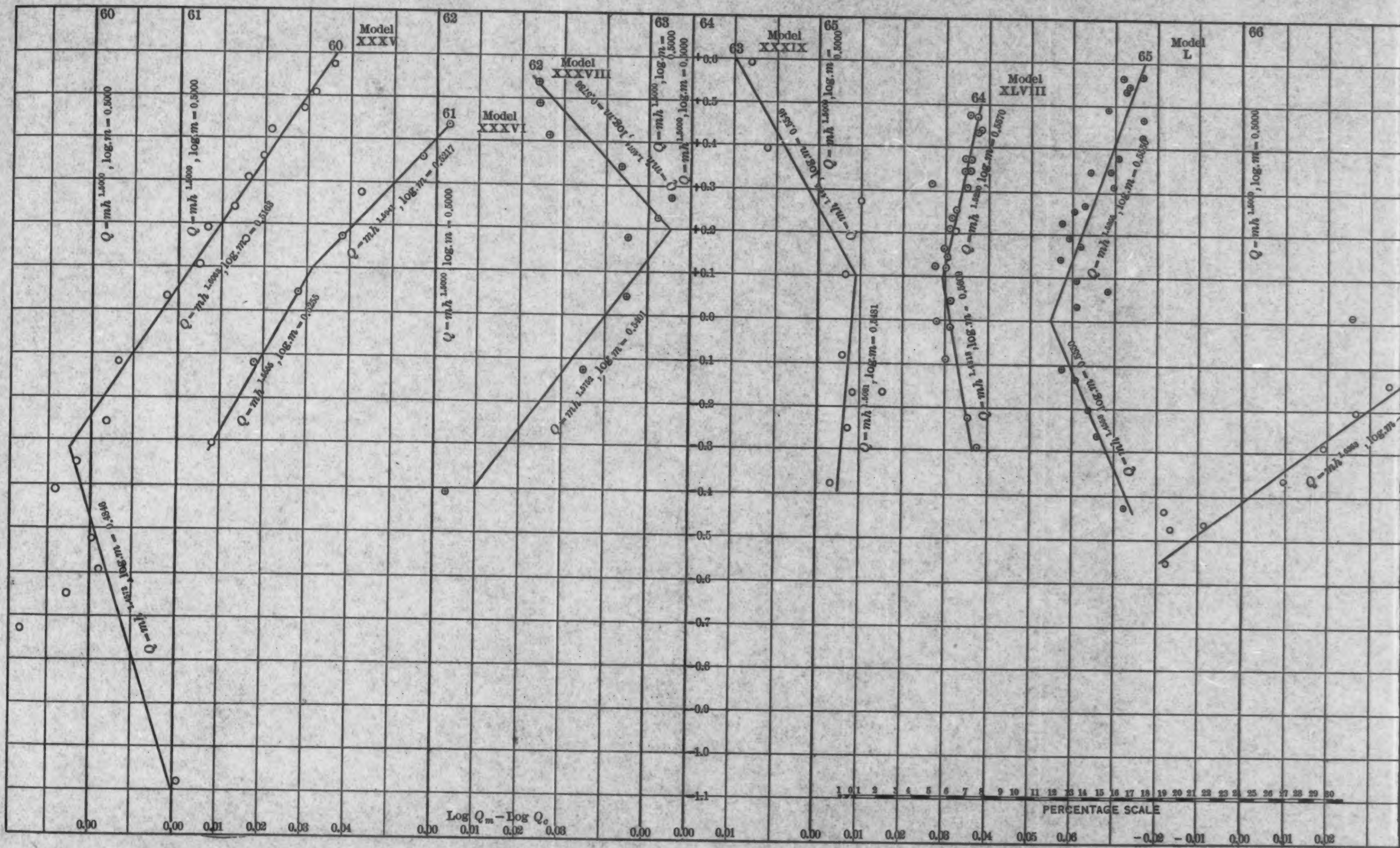


CORNELL MODEL XLIV.
FIG. 25.

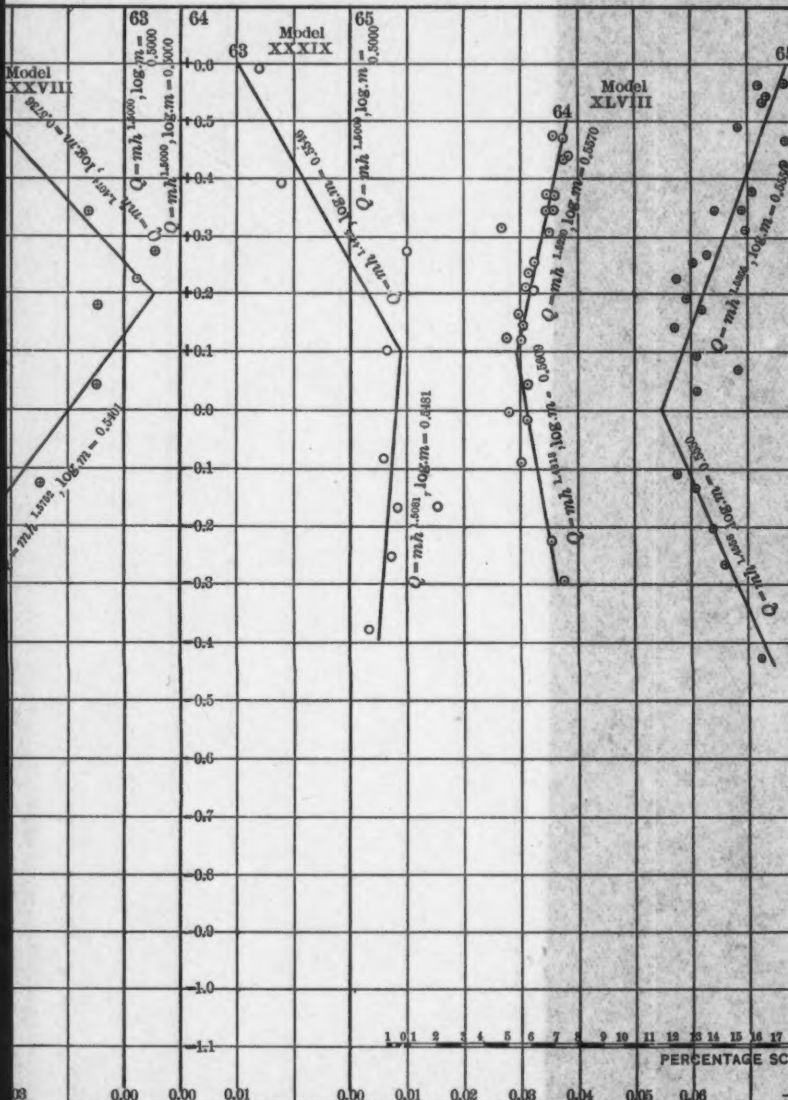
(1).—For Sharp Crests—Plates XXVII*a* and XXVII*b*.

The curves for experiments on sharp-crested weirs without end contractions are shown on Plates XXVII*a* and XXVII*b*. Although these "curves" are made up of right lines, it would be almost impossible to draw any line of any curvature which would fit these experiments better than do these adopted right lines.

On these plates, as arranged originally, an independent series of right lines was drawn, fitting the experiments with accuracy. These results, however, were not of great value in a general way, as one curve was applicable to only one set of experiments, and the scale on which the plates were made exaggerated the differences and led to confusion. For this reason, it was decided to make the lines or curves intersect in the same horizontal lines. The two lines, log. $h = -0.01550$ and log. $h = -0.3979$, were selected as those on which the lines representing the experiments should be broken. These lines indicate heads



Model XXXV
Model XXXVI
Model XXXVIII
Model XXXIX
Model XLVIII
Model L
60
61
62
63
64
65
66
$Q=mh^{1.5000}$, log.m = 0.5000
Log Q_m − Log Q_c
PERCENTAGE SCALE



Model XXXVIII
Model XXXIX
Model XLVIII
63
64
65
$Q = mh^{1.5000}$, log. m = 0.5000
$Q = mh^{1.5762}$, log. m = 0.5401
$Q = mh^{1.5081}$, log. m = 0.5481
$Q = mh^{1.5220}$, log. m = 0.5570
$Q = mh^{1.4818}$, log. m = 0.5609
+0.0
+0.5
+0.4
+0.3
+0.2
+0.1
0.0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
-0.7
-0.8
-0.9
-1.0
-1.1
PERCENTAGE SC
0.00
0.01
0.02
0.03
0.04
0.05
0.06

PLATE XXXII*b*.
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LYMAN ON
WEIR MEASUREMENT OF
STREAM FLOW.

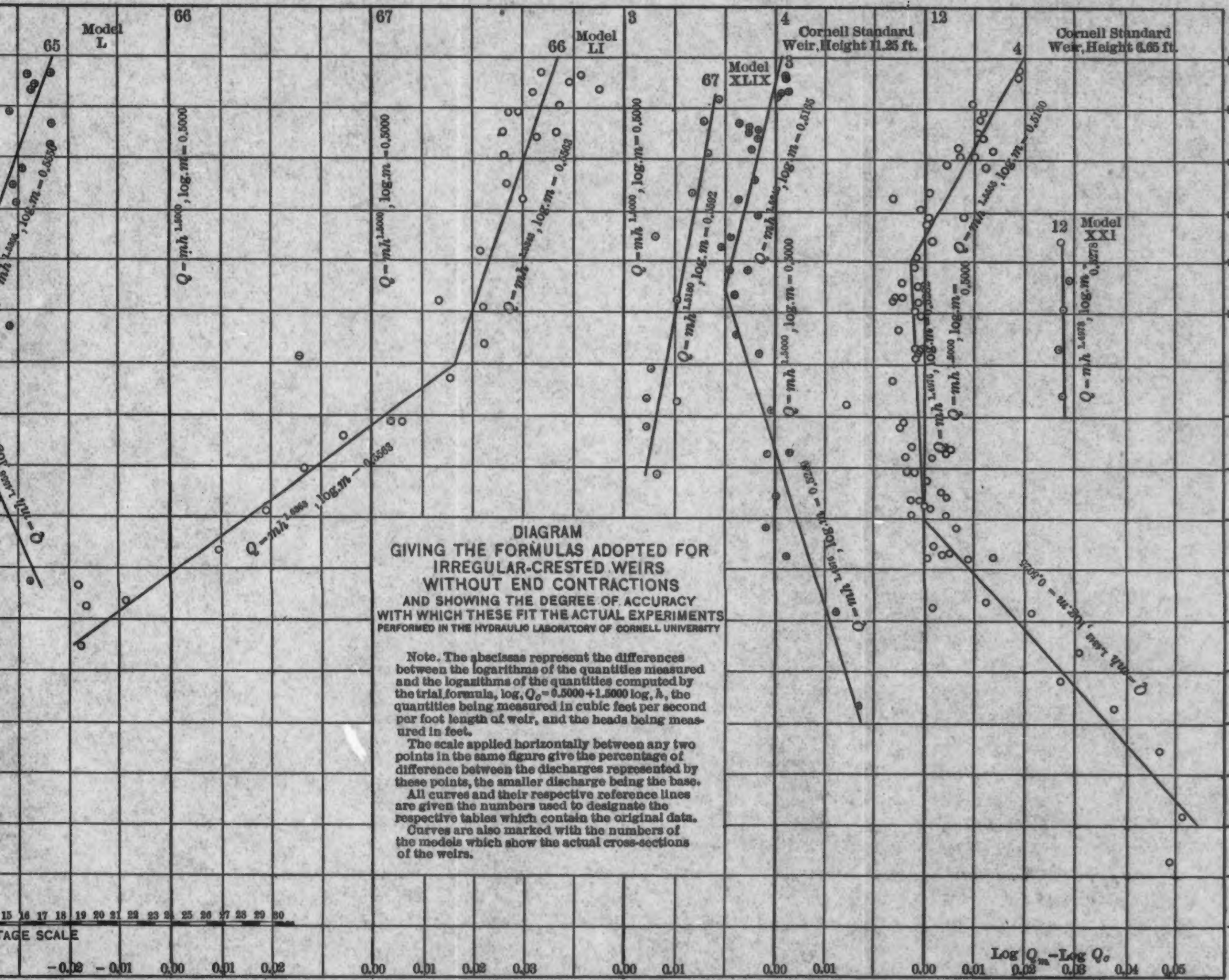


DIAGRAM
GIVING THE FORMULAS ADOPTED FOR
IRREGULAR-CRESTED WEIRS
WITHOUT END CONTRACTIONS
AND SHOWING THE DEGREE OF ACCURACY
WITH WHICH THESE FIT THE ACTUAL EXPERIMENTS
PERFORMED IN THE HYDRAULIC LABORATORY OF CORNELL UNIVERSITY

Note. The abscissas represent the differences between the logarithms of the quantities measured and the logarithms of the quantities computed by the trial formula, $\log. Q_o = 0.5000 + 1.5000 \log. h$, the quantities being measured in cubic feet per second per foot length of weir, and the heads being measured in feet.

The scale applied horizontally between any two points in the same figure give the percentage of difference between the discharges represented by these points, the smaller discharge being the base.

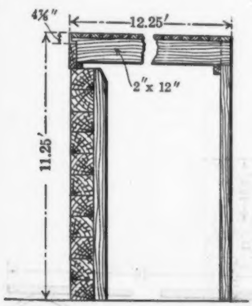
All curves and their respective reference lines are given the numbers used to designate the respective tables which contain the original data.

Curves are also marked with the numbers of the models which show the actual cross-sections of the weirs.

of 0.70 and 0.40 ft., respectively. It will be observed, however, that in the left-hand portion of Plate XXVII*a* five sets of plottings have been divided into four parts, each of which is represented accurately by a straight line. The additional line of division is log. $h = 0.3010$, or $h = 2.00$ ft. These results were easily compared and compiled in a general diagram.

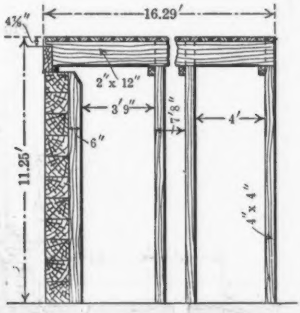
(2).—For Irregular Weirs and Broad-Crested Weirs—Plates XXX, XXXII*a*, and XXXII*b*.

Although there is not sufficient similarity in the cross-sections of the irregular weirs here shown, or in the results of the experiments on them, to warrant an attempt to devise a general diagram for such structures, there is a similarity in the results obtained from the experiments on broad-crested weirs which naturally produces the general diagram, Plate XXIV.



CORNELL MODEL XLII

FIG. 26.



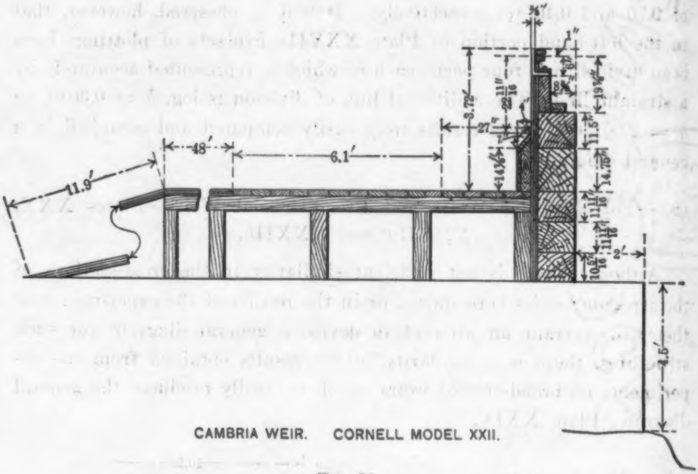
CORNELL MODEL XLIII AND XLIIIa.

FIG. 27.

H.—Use of the Scales on Plates XXVIIa, XXVIIb, XXX, XXXIIa, and XXXIIb, and Method of Constructing Them.

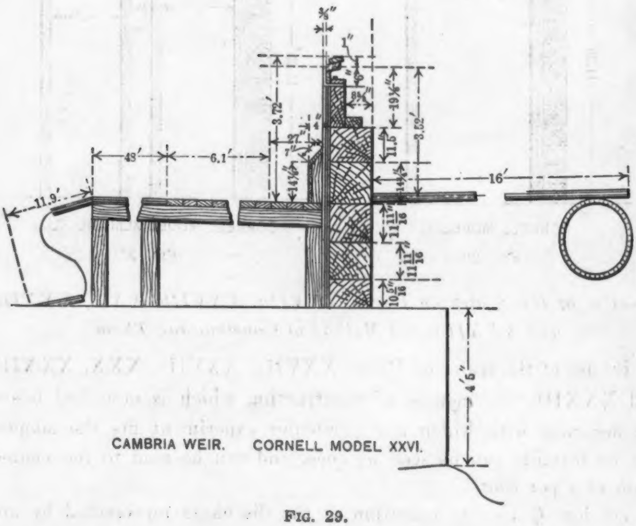
By use of the scales on Plates XXVII*a*, XXVII*b*, XXX, XXXII*a*, and XXXII*b*, the method of constructing which is described below, the accuracy with which any particular experiment fits the adopted line or formula can be seen at once, and can be read to the nearest tenth of 1 per cent.

Let log. Q be the logarithm of the discharge represented by any point on the diagram, and let p be any percentage of this discharge,



CAMBRIA WEIR. CORNELL MODEL XXII.

FIG. 28.



CAMBRIA WEIR. CORNELL MODEL XXVI.

FIG. 29.

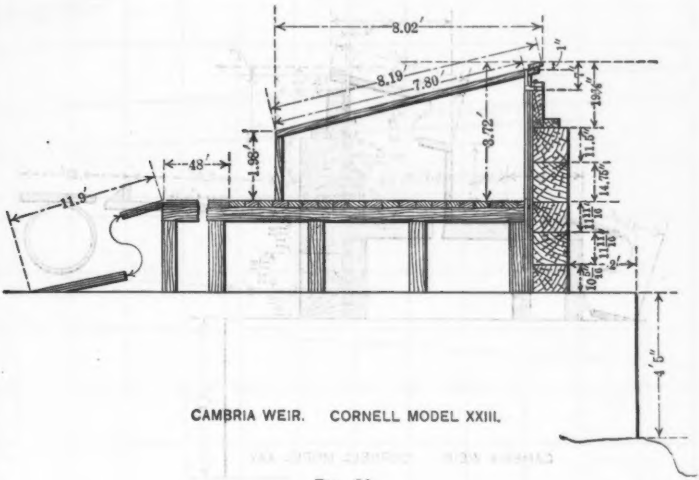

8.02′
8.19′
7.80′
1″
19⅝″
3.72′
1.98′
48′
11.9′
11.5″
14.75″
2′
4′5″
CAMBRIA WEIR. CORNELL MODEL XXIII.

FIG. 30.

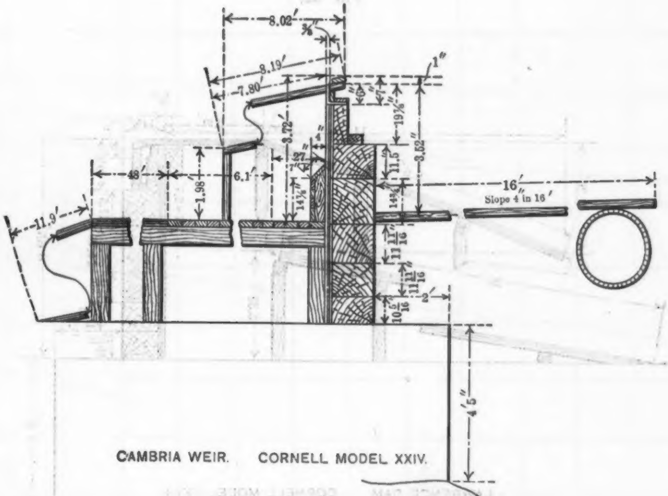
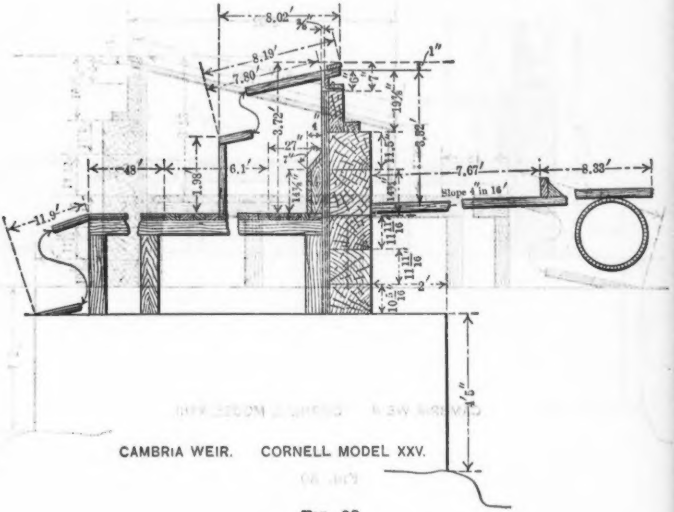
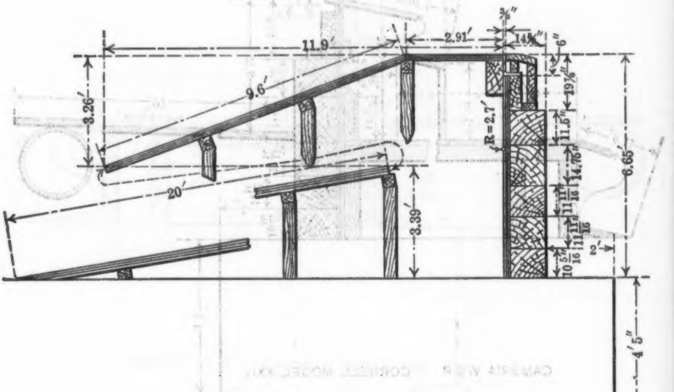

8.02′
8.19′
7.80′
1″
19⅝″
3.72′
3.52″
1.98′
48′
6.1′
11.9′
11.5″
16′
Slope 4 in 16′
2′
4′5″
CAMBRIA WEIR. CORNELL MODEL XXIV.

FIG. 31.



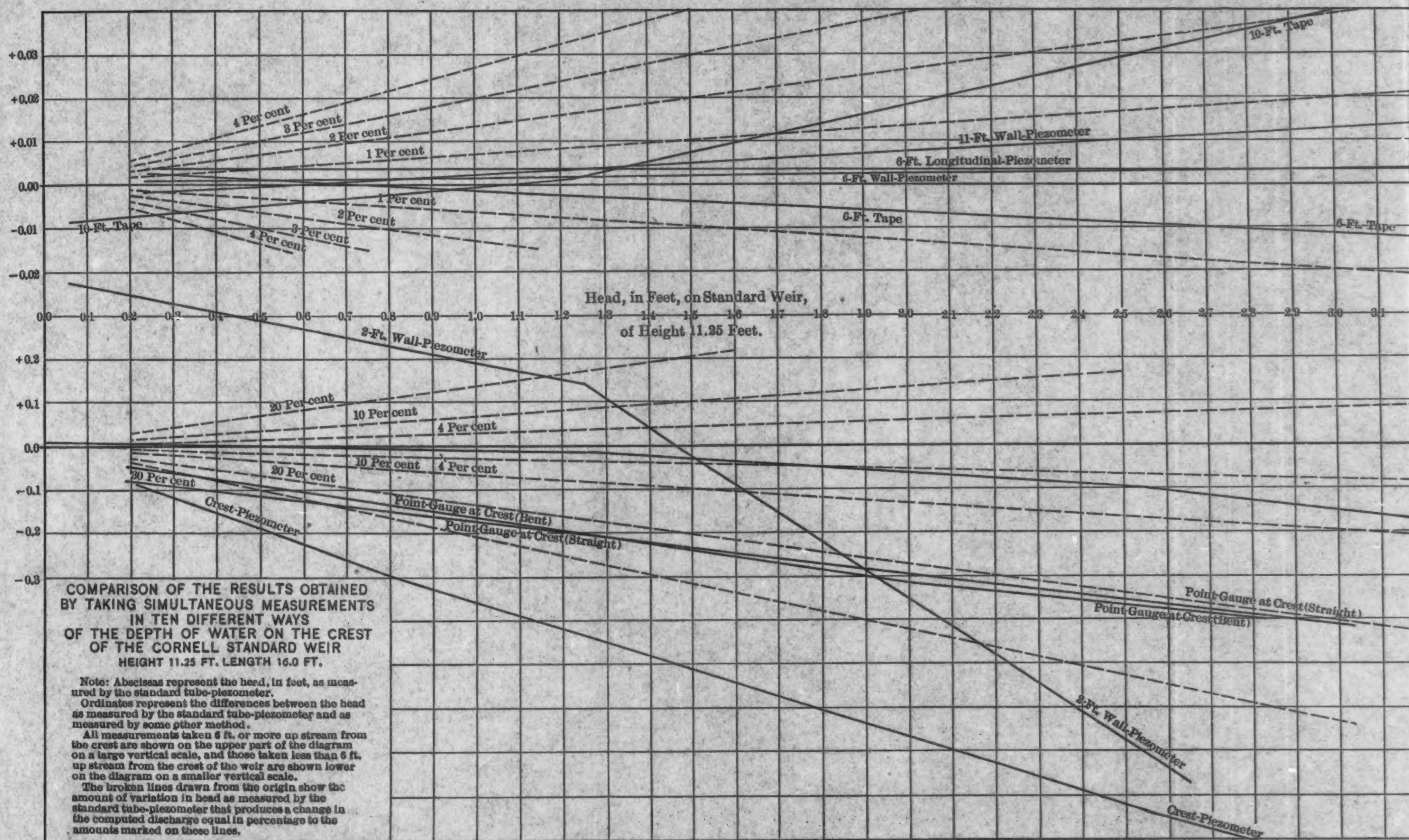
CAMBRIA WEIR. CORNELL MODEL XXV.

Fig. 32.



LAWRENCE DAM. CORNELL MODEL XXVII.

Fig. 33.



COMPARISON OF THE RESULTS OBTAINED BY TAKING SIMULTANEOUS MEASUREMENTS IN TEN DIFFERENT WAYS OF THE DEPTH OF WATER ON THE CREST OF THE CORNELL STANDARD WEIR

HEIGHT 11.25 FT. LENGTH 16.0 FT.

Note: Abscissas represent the head, in feet, as measured by the standard tube-piezometer.

Ordinates represent the differences between the head as measured by the standard tube-piezometer and as measured by some other method.

All measurements taken 6 ft. or more up stream from the crest are shown on the upper part of the diagram on a large vertical scale, and those taken less than 6 ft. up stream from the crest of the weir are shown lower on the diagram on a smaller vertical scale.

The broken lines drawn from the origin show the amount of variation in head as measured by the standard tube-piezometer that produces a change in the computed discharge equal in percentage to the amounts marked on these lines.

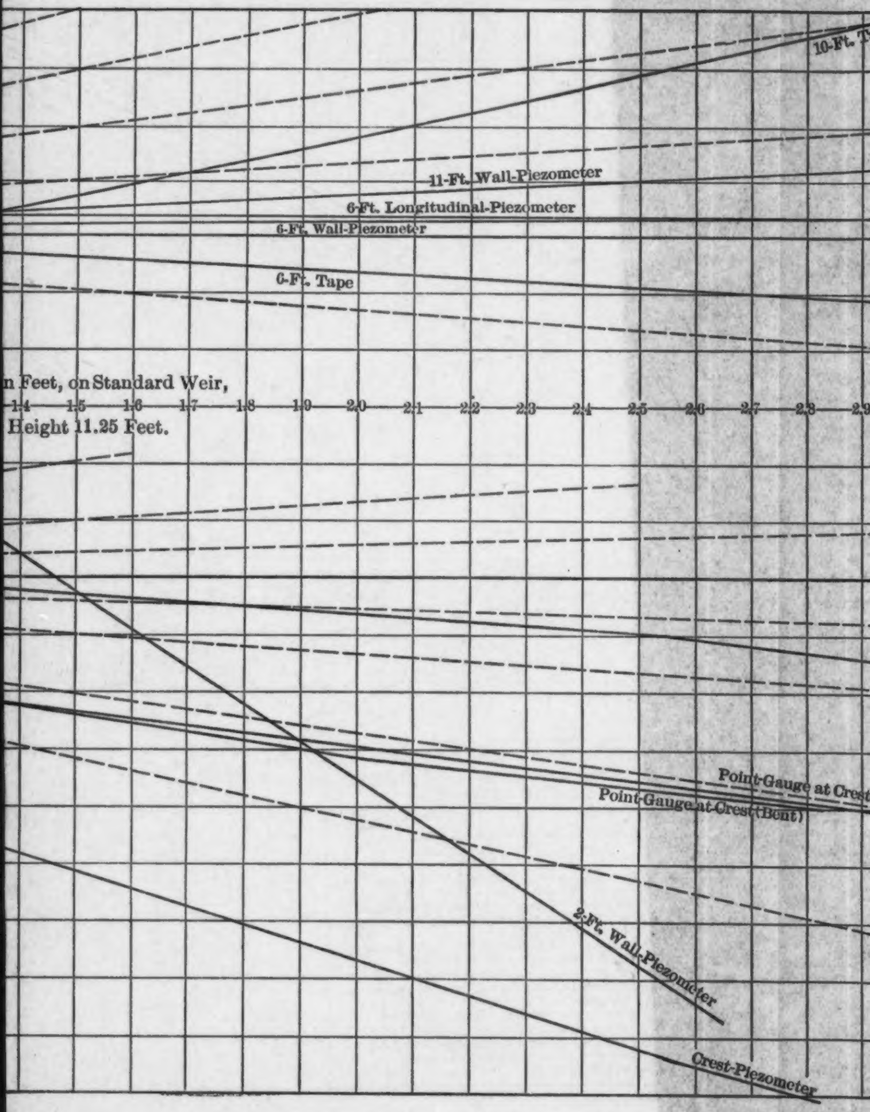

10-Ft. T
11-Ft. Wall-Piezometer
6-Ft. Longitudinal-Piezometer
6-Ft. Wall-Piezometer
6-Ft. Tape
n Feet, on Standard Weir,
1.4 1.5 1.6 1.7 1.8 1.9 2.0 2.1 2.2 2.3 2.4 2.5 2.6 2.7 2.8 2.9
Height 11.25 Feet.
Point-Gauge at Crest
Point-Gauge at Crest (Bent)
2-Ft. Wall-Piezometer
Crest-Piezometer

PLATE XXXIII.
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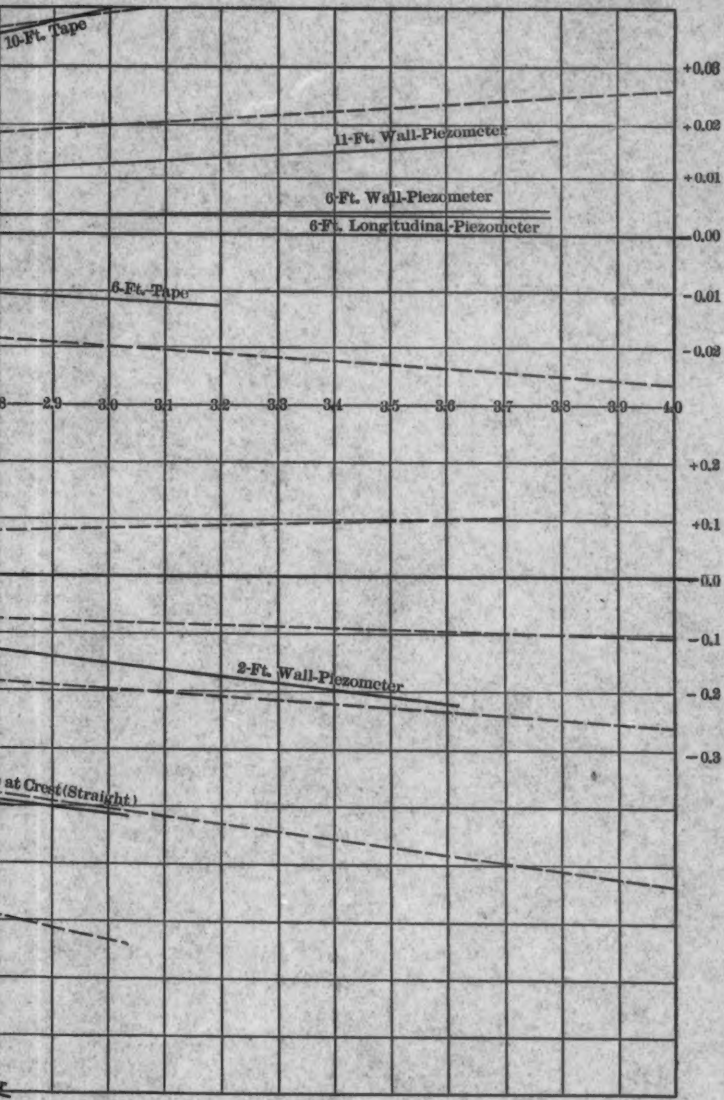
LYMAN ON
WEIR MEASUREMENT OF
STREAM FLOW.



Q. To find the distance, D, between the points, log. Q and log. $(Q + pQ)$:

$$D = \log.\ (Q + pQ) - \log.\ Q$$
$$= \log.\ [Q\ (1 + p)] - \log.\ Q = \log.\ Q + \log.\ (1 + p) - \log.\ Q$$
$$= \log.\ (1 + p).$$

Therefore, if p is 1%,

$$D = \log.\ 1.01 = 0.0043.$$

If p is 2%,

$$D = \log.\ 1.02 = 0.0086.$$

If p is 3%,

$$D = \log.\ 1.03 = 0.0128, \text{ etc., etc.}$$

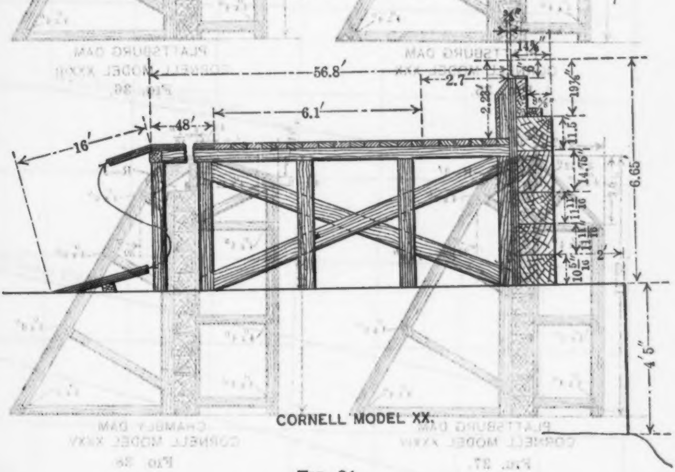


FIG. 34.

These values are plotted to make the scales drawn on each of these plates.

If p is negative, the quantity within the parentheses is less than log. Q, then

$$D = \log.\ Q - \log.\ (Q - pQ),$$

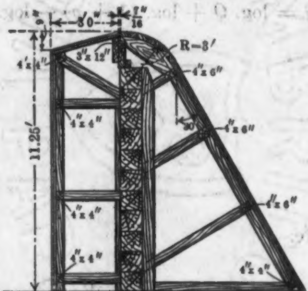
or $D = \log.\ (1 - p)$.

If $p = 1\%$, $D = \log.\ 0.99 = 9.9956 = -\ 0.0044.$

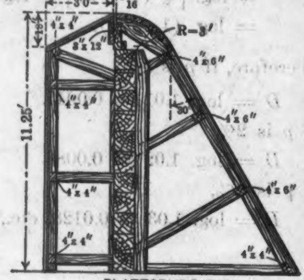
If $p = 2\%$, $D = \log.\ 0.98 = 9.9912 = -\ 0.0088.$

If $p = 3\%$, $D = \log.\ 0.97 = 9.9868 = -\ 0.0132.$

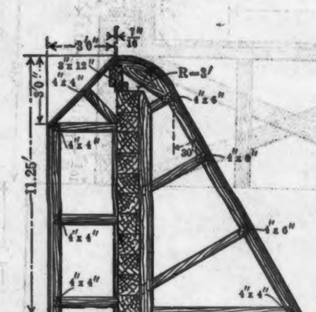
Therefore these scales, which are constructed from positive values of p, will only give correct results, technically, if the smaller quantity of discharge involved is used as the base.



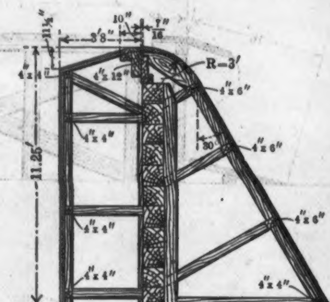
PLATTSBURG DAM.
CORNELL MODEL XXX.
FIG. 35.



PLATTSBURG DAM.
CORNELL MODEL XXXIII.
FIG. 36.



PLATTSBURG DAM.
CORNELL MODEL XXXIV.
FIG. 37.



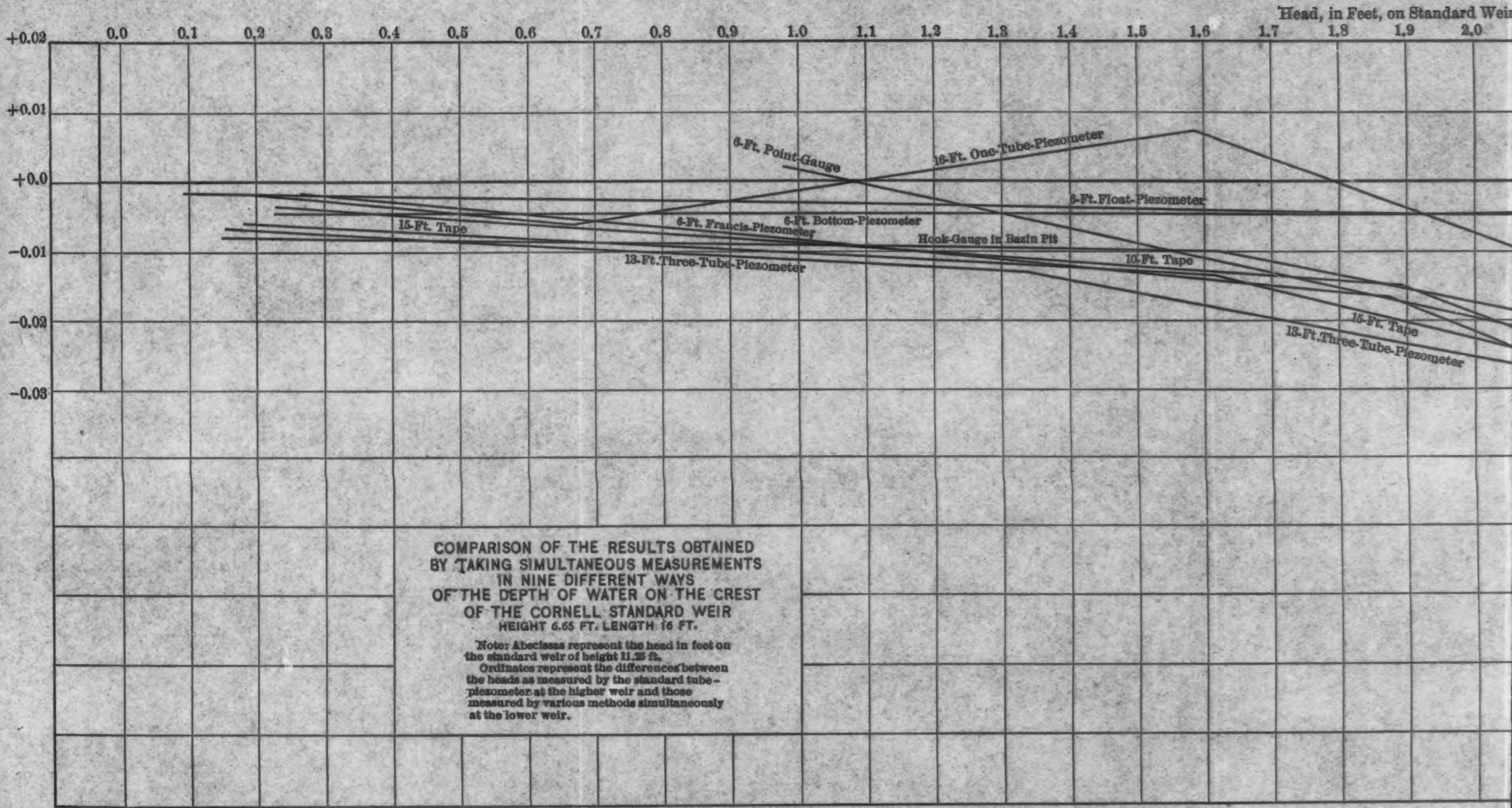
CHAMBLY DAM.
CORNELL MODEL XXXV.
FIG. 38.

I.—Method of Constructing the Final Diagrams.

(1).—General Statement.

Plates XXV, XXVI, XXVIII, XXIX, and XXXI are designed to show the relation that exists between the discharges of similar weirs.

To put the loci of the adopted equations of similar weirs on the same plate and to refer them to the same axes, it is only necessary to find two points on the locus of each equation and then draw that locus between the two lines that represent the logarithms of the limiting values of h to which the equation is to be applied.



COMPARISON OF THE RESULTS OBTAINED
BY TAKING SIMULTANEOUS MEASUREMENTS
IN NINE DIFFERENT WAYS
OF THE DEPTH OF WATER ON THE CREST
OF THE CORNELL STANDARD WEIR
HEIGHT 6.65 FT. LENGTH 16 FT.

Note: Abscissas represent the head in feet on the standard weir of height 11.25 ft.
Ordinates represent the differences between the heads as measured by the standard tube-piezometer at the higher weir and those measured by various methods simultaneously at the lower weir.

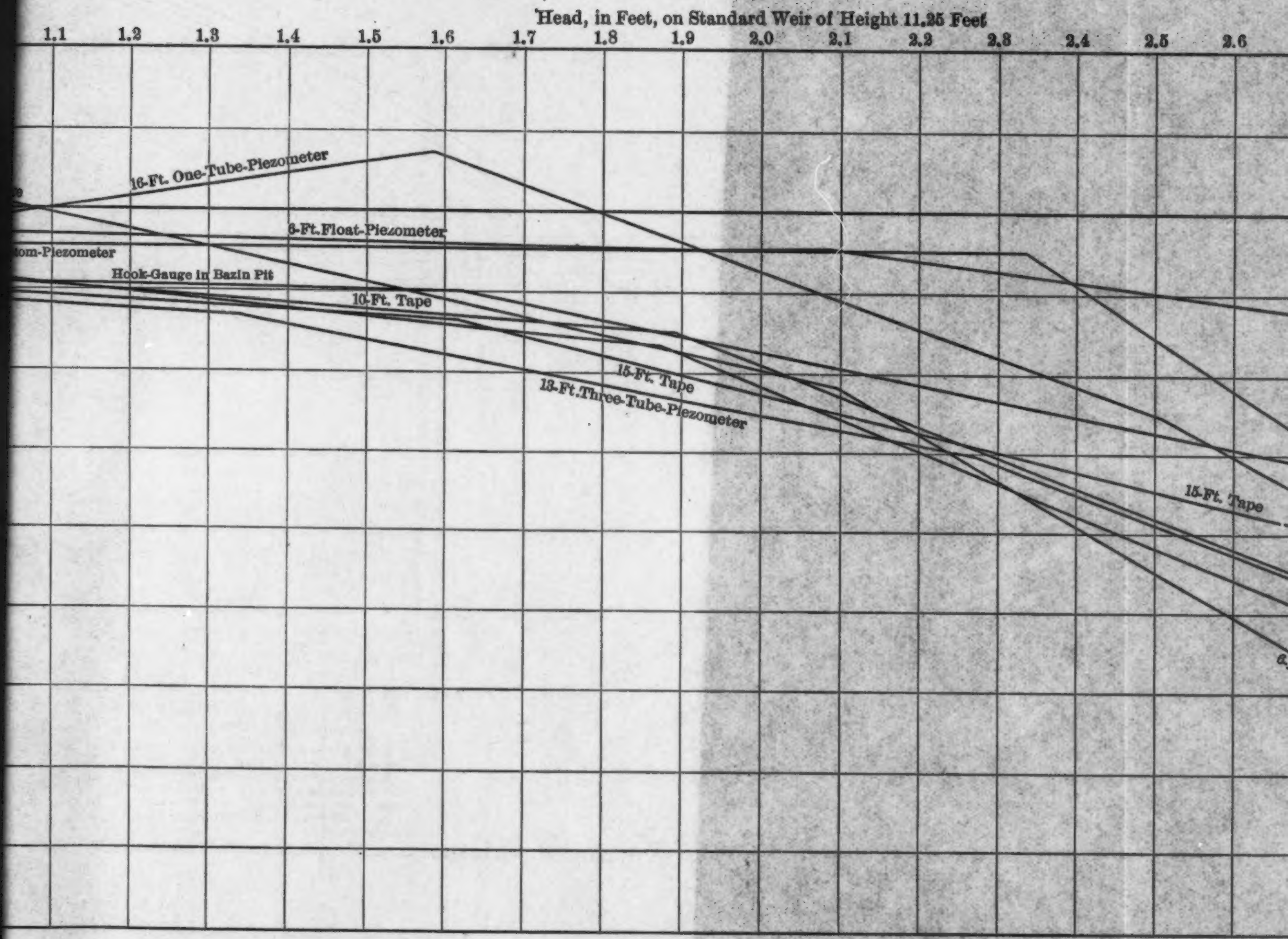

Head, in Feet, on Standard Weir of Height 11.25 Feet
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
1.9
2.0
2.1
2.2
2.3
2.4
2.5
2.6
16-Ft. One-Tube-Piezometer
6-Ft. Float-Piezometer
tom-Piezometer
Hook-Gauge in Bazin Pit
10-Ft. Tape
15-Ft. Tape
13-Ft. Three-Tube-Piezometer
15-Ft. Tape

PLATE XXXIV.
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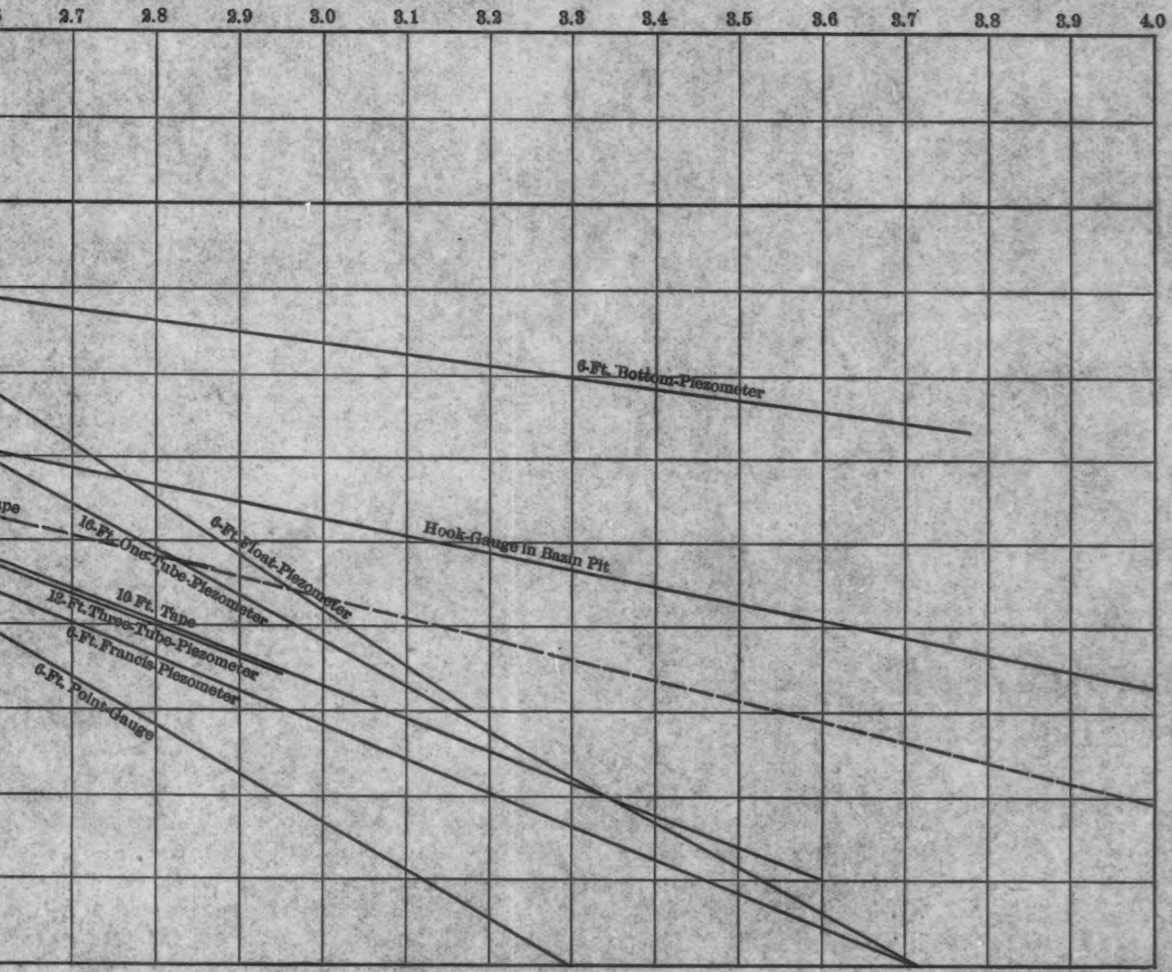
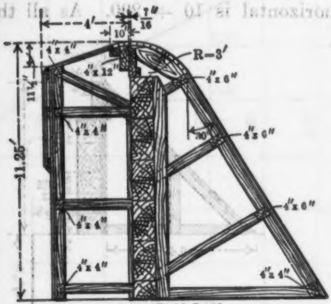
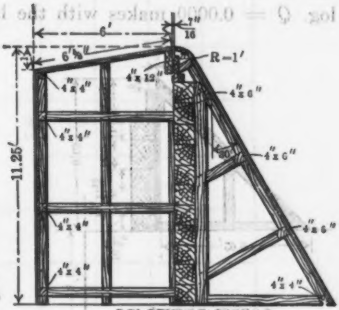
LYMAN ON
WEIR MEASUREMENT OF
STREAM FLOW.



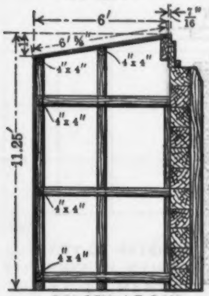
The two lines on which these points can be most easily located are the line, log. $h = 0$, and the line, log. $h = -1$. These two values of log. h, substituted in the equation for which the locus is desired, will give the two values of log. Q which locate the two points sought; but when these two values of log. Q have been found, one may be a plus 0.50 and the other a minus 0.95. If the horizontal scale desired



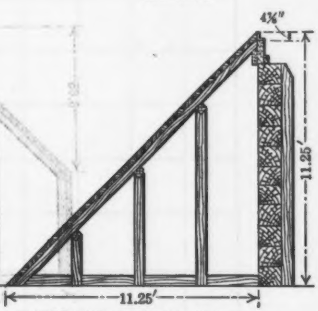
CHAMBLY DAM.
CORNELL MODEL XXXVI.
FIG. 39.



DOLGEVILLE DAM.
CORNELL MODEL XXXVIII.
FIG. 40.



DOLGEVILLE DAM.
CORNELL MODEL XXXIX.
FIG. 41.



CORNELL MODEL XLVIII.
FIG. 42.

is that used on the original plotting of Plate XXIX, namely, 2 in. for each unit in the second decimal place of the logarithm, the plus 0.50 is 100 in. to the right of the origin and the minus 0.90 is 190 in. to the left of it, thus requiring the drawing to be 290 in. long, in order to contain the part of the locus between the two lines, log. $h = 0$ and log. $h = -1$. If, however, the zero on the line, log. $h = 0$, be brought to the left until the point, plus 0.50, is directly over the minus 0.90,

then the line drawn, instead of making a very small angle with the horizontal, will be vertical, and the line, log. $Q = 0$, will make a small angle with the horizontal.

In this new position the zeros of the two reference lines, log. $h = 0.0000$ and log. $h = -1.0000$, are 290 in. apart horizontally and 10 in. apart vertically, or the tangent of the angle that the line, log. $Q = 0.0000$ makes with the horizontal is $10 \div 290$. As all the

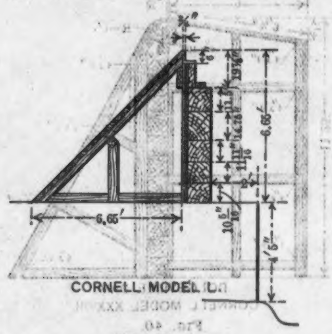


FIG. 43.

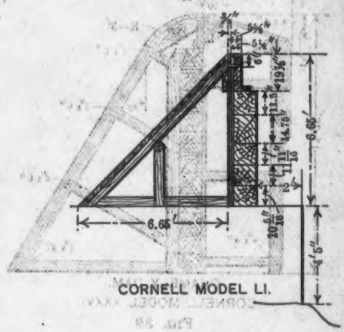
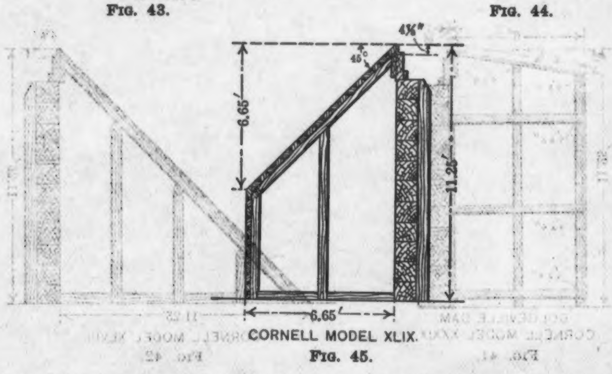


FIG. 44.



CORNELL MODEL XLIX.

FIG. 45.

log. Q lines are parallel, this is the tangent of the angle which all these lines make with the horizontal. Knowing this angle, if one point on any log. Q line be found, the line can be drawn.

Let the horizontal line, ab, in Fig. B, Plate XXVIIb, represent the line, log. $h = 0.00$, and let the horizontal line, ed, represent the line, log. $h = -1.00$. The zeros of these two lines are as shown, and the line joining them is the line, log. $Q = 0.0000$. Let cb be any

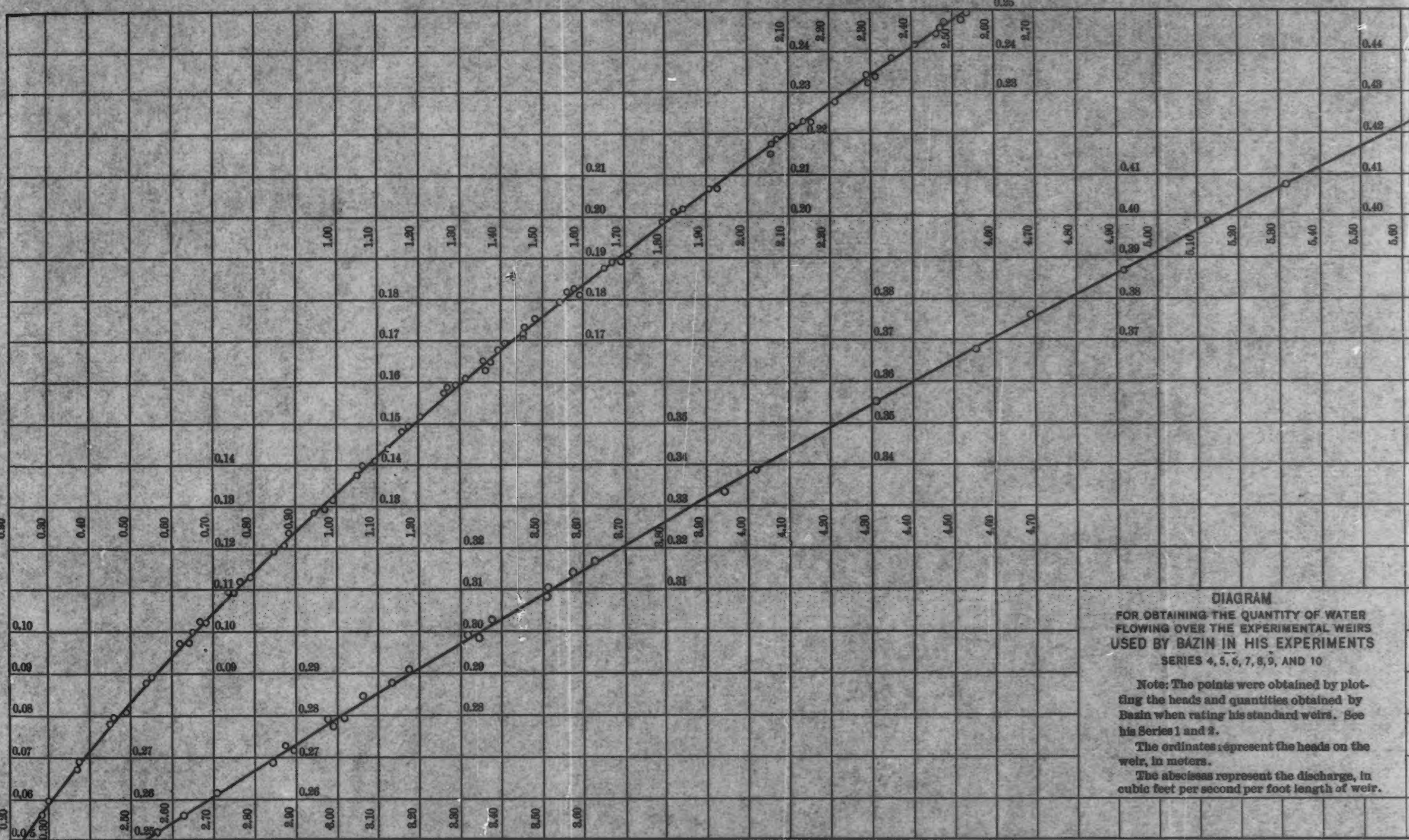


DIAGRAM
FOR OBTAINING THE QUANTITY OF WATER FLOWING OVER THE EXPERIMENTAL WEIRS USED BY BAZIN IN HIS EXPERIMENTS
SERIES 4, 5, 6, 7, 8, 9, AND 10

Note: The points were obtained by plotting the heads and quantities obtained by Bazin when rating his standard weirs. See his Series 1 and 2.

The ordinates represent the heads on the weir, in meters.

The abscissas represent the discharge, in cubic feet per second per foot length of weir.

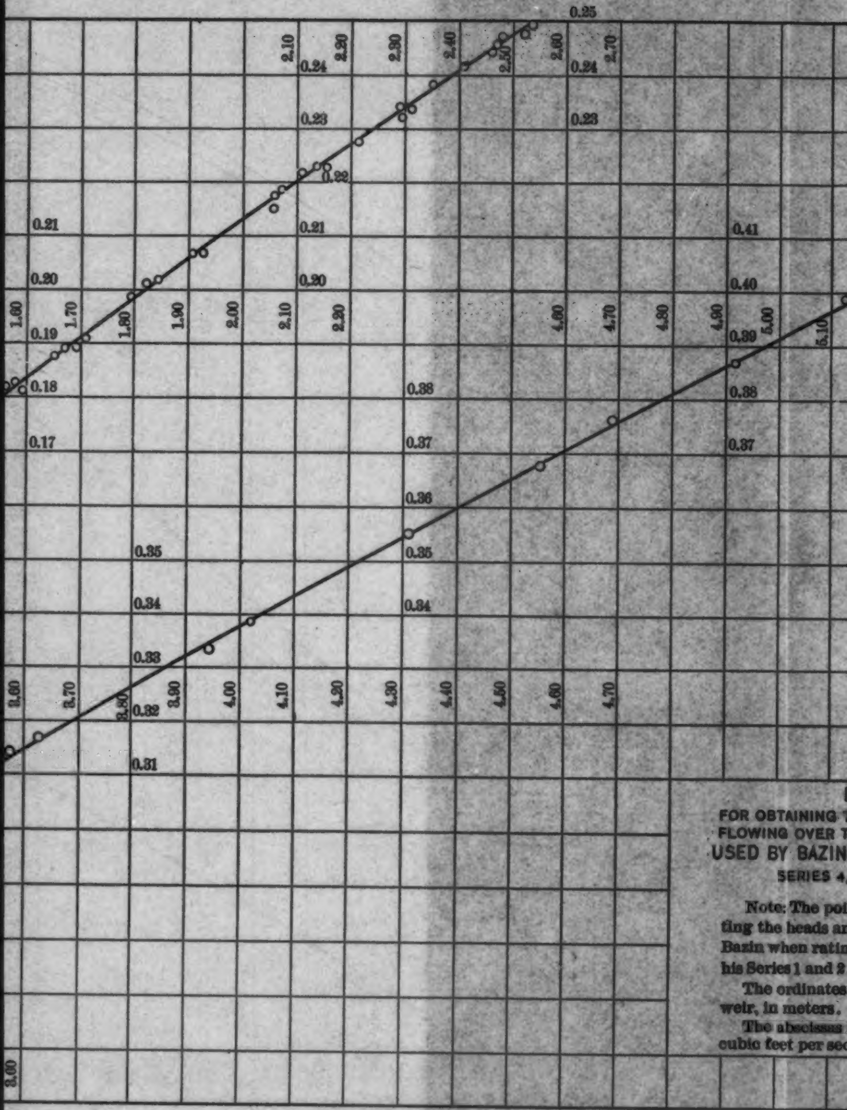


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PLATE XXXV.
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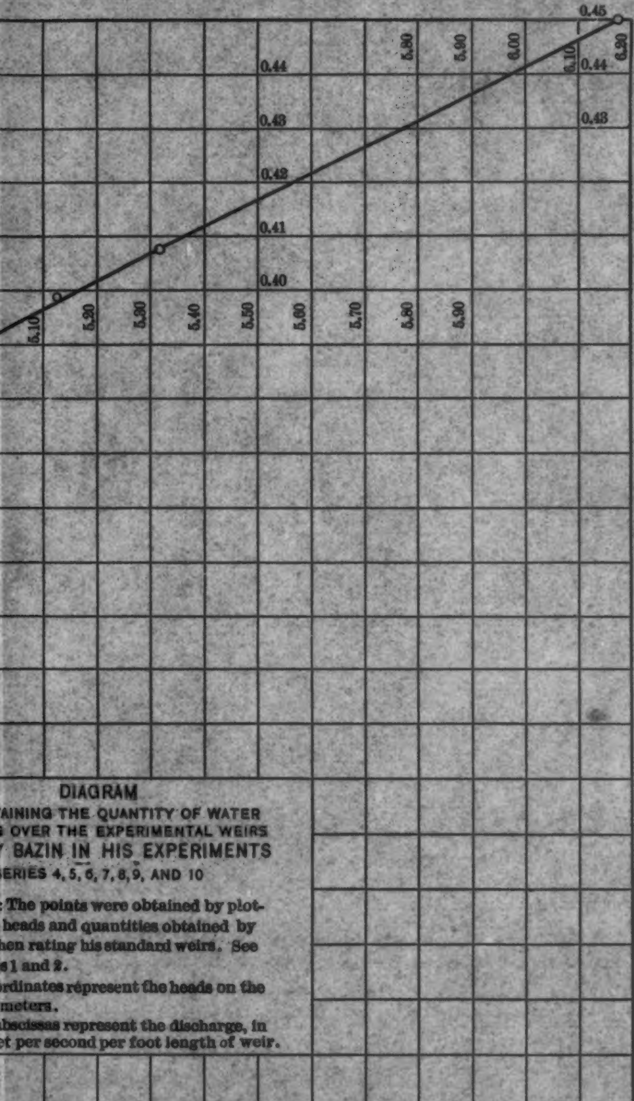
WEIR MEASUREMENT OF
STREAM FLOW.



DIAGRAM
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other log. Q line. It is required to locate this line on the drawing. The location of the point, b, is known from the value of the logarithm of the quantity which the line, cb, is to represent; but this point may not fall within the limits of the drawing. However, knowing the tangent of the angle the line makes with the horizontal, and the location of the point, b, in which it intersects the horizontal line, ab, any two points on the line, cb, can be located with the aid of a simple proportion, and the required part of the line can then be drawn. In the same way any number of quantity lines may be constructed.

Lines representing the heads are readily located, as they are drawn horizontally through points on the axis which represent the logarithms of the heads it is desired thus to indicate.

By similar methods, Plates XXV, XXVI, XXVIII, and XXXI have been constructed. The co-ordinates have been placed at a different angle from that used in the construction of Plate XXIX. For Plate XXVIII, the point, + 0.40, has been placed over — 0.78, so that the zeros are 118 in. apart horizontally. For Plates XXV, XXVI, and XXXI, the point, + 0.40, has been placed over — 0.90, so that the zeros are 130 in. apart horizontally.

(2).—For Sharp-Crested Weirs.

(*a*).—*Discharge Over Specific Weirs Used in Experiments.*—The loci of the adopted equations for these weirs are shown on Plates XXVII*a* and XXVII*b*. The constants in these equations, with the limits of head between which they apply, are given in Table C.

For comparison, the curves are brought together on the same axes on Plates XXVIII and XXIX, which are identical except for scale and angle of co-ordinates.

It will be seen that these curves are all more or less similar, and that the changes in form, and in position with respect to the locus of the trial equation, are, in general, proportional to the height of the weir.

(*b*).—*Effect of Friction in Channel of Approach.*—How well justified Bazin's assertion is that from the results obtained he could find no difference in the discharge per foot of length of his standard weir of height 3.72 ft., when the length was 6.56 ft. and when the length was 3.28 ft., is shown by the two curves for these weirs. The shorter weir gives the greater discharges for heads near 0.40 ft., and for higher heads the discharges are practically the same.

The small discharges given by Bazin's formula for heads below 0.5 ft. when applied to the Cornell weirs justifies the assertion repeatedly made by Professor Williams that the experiments on these weirs with low heads are not reliable.

(*c*).—*Diagram for Discharge Over Any Weir.*—In order to make a general diagram for weirs of this class, the horizontal distances between the locus of the trial formula (log. $Q = 1.5000$ log. $h + 0.5000$) and the loci of the adopted formulas were read on Plates XXVII*a* and XXVII*b* on the four lines, log. $h = -0.1000$, log. $h = -0.1550$, log. $h = -0.3979$, and log. $h = -0.7000$.

The two inner of these four lines were selected because, as previously explained, all the curves have been made to intersect in them. The other two were selected as convenient lines, one being near the upper, and the other near the lower, limit of the diagram.

The values thus read were plotted as abscissas and the heights of the weirs as ordinates on Fig. 14. From the curves which these points determine, mean values were taken for weirs of assumed heights.

The values thus obtained were laid off on the four corresponding lines on Plate XXI as originally constructed on cross-section paper. The locus of the trial equation, of course, was first laid off on the diagram. Only that portion of the original diagram necessary to show the relation existing between the heads, the discharges, and the heights of the weirs, has been reproduced on Plate XXI; it is a general diagram for measuring the water flowing over all weirs of this class.

Plates XXIII, XXIV, XXV, XXVI, XXVIII, and XXXI are also reproductions of only those portions of the original drawings which are really necessary to show the relation between the quantities named thereon.

(3).—For Broad-Crested Weirs.

On Plate XXXI are brought together all the curves for broad-crested weirs, the widths varying from $\frac{3}{8}$ in. to 16.29 ft. All these weirs have a height of 11.25 ft. The curves marked Cornell Ht. 6.65 and Table 5 are for weirs of width $\frac{3}{8}$ in. and height 6.65 ft. The weirs with the narrower crests are, for all practical purposes, weirs with sharp crests.

It will be noted that, up to a certain point, that is, up to a certain head, the curves for all these weirs practically coincide; the point at which this coincidence ceases varies with the width of the weir. For

the Cornell Model XL, which has a width of 5¾ in., the curves leave the "broad-crest" line at a head of 0.36 ft. and extend across the diagram diagonally, reaching the "sharp-crest" line at a head of 0.80 ft. This is the head at which the sheet jumps entirely over the "broad crest"; for all greater heads this weir operates as if it had a sharp crest, and its curve, above this point, coincides with that for sharp crests.

As with the weir having a crest 5¾ in. wide, so is it, in a general way, according to this diagram, with those of other widths. The "broad-crest" line is practically parallel to the "sharp-crest" line. The curves for all broad crests follow the "broad-crest" line up to a certain head; then they jump in a straight line diagonally across the diagram to the "sharp-crest" line, and follow it up to the limit of the experiments.

As explained previously, this may be accounted for as follows: As the head increases, a definite point is reached at which a vacuum starts to form under the sheet on top of the crest, with the effect of increasing the flow. This effect is increased until, at a certain head, the sheet jumps entirely free from the crest, and then for all higher heads the weir acts as if the crest were "sharp."

Mean values or lines for the "broad-crest" condition, the "sharp-crest" condition, and the "intermediate" condition, are shown as broken lines on Plate XXXI; and these form the whole diagram on Plate XXIV, which is used for obtaining the discharge over all weirs of this height with broad crests.

Fig. 15, containing the information from which Plate XXIV was drawn, shows the relation between the widths of the crests and the heads at which the changes from the broad-crest condition begin, and those at which the sharp-crest condition is reached. From the two curves on this plate, the head can be read at which the broad-crest condition ceases for a crest of any width, within the limits of the experiments, and that also at which the sharp-crest condition begins. From these curves, information was secured for constructing Plate XXIV, which is the diagram intended to be used in practice for obtaining the discharge over any weir with a broad crest having a height of 11.25 ft., within the limits shown on this plate.

It might naturally be expected that for some heads the flow over a broad-crested weir would be greater than that over a similar sharp-crested weir, just as the flow through a short tube is greater than

that through a sharp-edged orifice of the same diameter under similar conditions; but the curves on Plate XXIV show conclusively that this is not the case for crests wider than 5⅜ in.

This assertion might be made concerning all broad crests, if it were not for the fact that Model XXII gives a greater discharge than Model XXI, the latter having the narrower crest. Notwithstanding this case, it appears that, for weirs with broad crests, the friction on the crest is a more important factor in the discharge than is the tendency of the broad crest to decrease the effect of contraction.

(4).—For Irregular Weirs with Cross-Sections of Right Lines.

The curves for the weirs of this class have been brought together on Plate XXV. The two curves marked XXVII*a* and XXVII*b* are the results of experiments on a model of the Lawrence Dam* built over the upper standard weir in the Hydraulic Laboratory of Cornell University. The curve marked XXVII *a* is the result of plotting the heads as they were measured by the 15-ft. tape. The curve marked XXVII *b* is the result of plotting the heads measured by the 20-ft. tape. It may be that the shape of the dam produced a slight swell in the surface of the water 15 ft. up stream, thus indicating a smaller discharge for the same head than is shown by the 20-ft. tape.

From Plate XXVI and the plate now under consideration, it appears that very slight differences in models make comparatively large differences in the discharges over them. Observe, for example, the great difference between the two curves, XXI and XXII, on this plate. One is the curve for a weir with a sharp crest and a vertical up-stream face 3.65 ft. high, and the other is the curve for the same weir with the base of the bulkhead thickened 4 in. on the up-stream side for a height of 14⅞ in. from the bottom. The height of this weir was increased 1 in. by timbers attached on the down-stream side of the crest.

For a head of 0.42 ft., Model XXII gives a discharge about 3.6% greater than Model XXI, and for a head of 0.9 ft., it gives a discharge about 7.7% greater.

The constants for the equations for these weirs are given in Table E.

* "Lowell Hydraulic Experiments," p. 136.

(5).—For Irregular Weirs with Cross-Sections of Right Lines and Curves.

The curves for the models of this class are presented on Plate XXVI. It is interesting to see how uniformly, for high heads, these models give greater discharges than weirs with sharp crests, and that the opposite is true for low heads. Model XXXIV is an extreme example. Its discharge is greater than that of the sharp-crested weir by more than 17% for a head of 3 ft. and, for a head of 0.65 ft., the discharge is about 3.3% smaller. The two curves, XXXV and XXXVI, show what a material difference a small change in the form of a model may make in the quantity of water which will flow over it under a given head.

The constants for the equation for these weirs are given in Table F.

II.—Comparison of Various Methods of Measuring the Depth of Water on the Crest of a Weir.

A.—*General Remarks.*

All the data presented under this heading have been obtained from experiments conducted under the direction of Professor Williams, in the Hydraulic Laboratory of Cornell University. These experiments, as far as the writer knows, comprise the most extensive set of this nature which has yet been made. Every detail of the investigation was watched with great care.

(1).—On Measuring Heads.

Each of the readings used for obtaining the head on the weir is the average of a series of readings taken during a period of from 15 to 30 min., during which time the head was kept as nearly constant as possible, and readings were taken at rates varying from two to six per minute.

One of these periods is called a run. The heads, in each of these runs, as measured by the standard tube-piezometer, have been plotted as abscissas, with the heads, as measured by the other instruments, as ordinates. The scale used is one gauge-scale unit, or one double centimeter, to the inch. The centers of gravity of the curves thus drawn were found first as a whole, then for each of the two halves of the curve, sometimes for the four fourths, and in a few cases for the eight eighths. Then, by passing lines analytically through some

or all of the points thus found, equations have been obtained which express the relation between the heads as accurately as that relation could be seen on the plottings.

These plottings, these equations, and their derivations, form a part of a thesis on "The Flow of Water Over Weirs" presented to Cornell University by the writer in 1903 for the M. C. E. degree. This matter is too extensive, however, to be reproduced in detail in this paper; but the heads as measured are given in Table A, and the results obtained are presented on Plates XXXIII and XXXIV. These plates show how widely the obtained results may differ if the depth of water on the crest of a weir is measured in different ways, even though generally accepted by hydraulicians as methods which give correct results.

It is generally understood that, for computing the discharge of a weir, the head should not be measured less than 6 ft. up stream from the crest; therefore, the results obtained by measurements taken 6 ft. or more above the crest are shown independently in the upper part of Plate XXXIII. The abscissas represent the heads, as measured by the standard tube-piezometer, plotted originally on a scale of 1 in. to 0.1 ft., and the ordinates represent the differences between the standard-tube heads and those measured by some other method plotted originally on a scale of 1 in. to 0.01 ft.

The differences between the heads as measured by the standard-tube and those measured less than 6 ft. up stream from the crest of the weir are so great that, in order to show them within the limits of the plate, the vertical scale had to be reduced to the scale shown in the lower portion of the drawing.

(2).—On the Piezometer.

Before taking up in detail descriptions of the different measuring devices used in securing the data herewith presented, it may be well to state that, in general, the apertures of a piezometer are supposed to be made in a surface or in surfaces the elements of which are parallel to the direction of the line of flow of the water. The water flowing through these openings is conducted by a water-tight pipe to a vertical glass tube securely fastened to a framework. Next to the tube, and back of it, there is a carefully graduated scale, on which, with the aid of a rider, the elevation of the water can be read; this elevation is assumed to be the same as that of the surface of the water which covers the openings of the piezometer.

The capillary action of the water makes the surface of the water column in the glass tube assume the form of a meniscus with its convex side downward. As the lowest point of this surface is definite and easily seen, piezometer readings were taken by placing the index of the rider in the horizontal plane containing this lowest point. When the rider is thus placed, its zero is at the reading for that column of water.

(3).—On the Tape.

A "tape," as used in the hydraulic laboratory for measuring the depth of water on the crest of a weir, consists of a carefully graduated steel tape, with a plummet attached to it in such a way that the distance from the end of the tape to the point of the bob is constant. This tape, with the plummet attached, is hung over the end of a gradually, but not very sharply, rounded block, on top of which is a brass plate having a groove of proper width and depth to receive the tape. An index line, used to mark the reading on the tape, is made on the upper surface of this plate at right angles to the sides of the groove. When the point of the plummet is just touching the water, this index line is at the reading of the tape for that particular elevation of the water surface.

(4).—On the Point- or Hook-Gauge.

A word may be added here also to explain the method used when readings are taken with a point-gauge or a hook-gauge. Two blocks, some 5 ft. apart, and containing rectangular notches, are placed so that one is vertically above and symmetrically situated with respect to the other. These are used to keep the gauge in a vertical position, and a bolt-head or some other permanent mark, properly situated, receives the small bar or rest on the back of the rod, thus fixing the position of the index on the rod. The point, which is attached to the sliding portion of the rod, is moved vertically until it is in contact with the surface of the water. When thus placed, the index of the rod marks the gauge reading for the depth of water which is being observed.

(5).—On the Method of Getting Zeros.

The readings of these various instruments for various elevations of the water surface, are of value only when the height of the water surface, with respect to some particular and well-defined datum, can

be found from them. In weir work this datum is the horizontal plane containing the crest of the weir. The readings of these various instruments, when the water surface in the channel of approach coincides with this datum, are called the "zeros" of the various tapes, piezometers, or gauges.

For determining the zeros of the various measuring devices used, a hook-gauge was clamped to the crest of the weir on which heads were to be measured, care being taken to have the instrument attached in such a way that the movable upright bar was in a vertical position. The accuracy of this adjustment was tested by sighting in two directions at right angles with each other along the side of a plummet string held at arm's length. When this test showed the rod to be vertical, the point of the hook was, with great care, brought into the horizontal plane containing the crest of the weir by turning the screw on the instrument to which the vertical rod is clamped. When a small, carefully-tested pocket-level, placed with one end on the point of the hook and the other on the crest of the weir, showed the point to be on a level with the crest, the reading of the gauge was taken and recorded. The difference between this reading and any subsequent reading, taken with the point of the hook at the surface of the water, was the distance of the surface of the water below the crest of the weir. After making a careful comparison of the times indicated by the different watches used, and recording the results of this comparison, the observations were begun. During the observations the surface of the water in the channel of approach was kept but a short distance below the crest. Readings on the different measuring devices were taken as rapidly as they could be made with care, and these, with the time of each observation to the nearest second, were recorded. As it was practically impossible to keep the elevation of the water surface constant, several series of readings were taken in the manner just described, some with the surface of the water slowly rising, and others with the surface of the water slowly falling. At the close of each series of observations the times were again compared, and the reading of the hook-gauge, with the point of the hook in the horizontal plane containing the crest of the weir, was carefully checked.

By plotting these readings, using the times as abscissas and the readings as ordinates, curves were drawn showing the readings that any measuring device in use would give, if a reading had been taken

at any moment. Two times were selected, one near the commencement, the other near the close, of each run or series of observations, and the readings of the various instruments at the times selected were read from the curves. The distance of the water surface below the crest of the weir was the difference, as thus indicated, between the "zero" reading of the hook-gauge and the hook-gauge reading at the time selected. A correction equal to this difference, properly applied to the simultaneous readings of the other instruments, gave their respective zeros. The differences between the means of the zeros thus found and readings taken during the subsequent experiments were the heads required.

(6).—On Expressions Used.

Oftentimes, in this paper, by the reading of a tape, a gauge, or a piezometer, is meant the head of water on the weir as measured with the instrument named, and by the head of water on the weir is meant the vertical distance, in feet, between the horizontal plane containing the crest of the weir and the plane surface of the water in the channel of approach.

The different piezometers, tapes, and gauges considered have been called by names that tell, first, approximately how far up stream the head was measured, and, secondly, what the measuring instrument was.

B.—Measurements of Heads at the Standard Weir.

(Height 11.25 Ft.)

(1).—With the Standard Tube-Piezometer.

The standard tube-piezometer consists of the following: Three $\frac{3}{4}$-in. pipes, each 4 ft. long, and perforated with four rows of $\frac{3}{16}$-in. holes, each row consisting of 12 holes 1 in. apart, longitudinally, and 90° apart circumferentially, the perforated portion being near the down-stream end. These three pipes are as nearly horizontal as may be, and are parallel to the walls of the channel. Each pipe is supported by two legs or pipes attached to its ends, one by a 90° elbow, the other by a T, the legs themselves being fastened to a base of two transverse and three longitudinal pipes resting on the bottom of the canal. The down-stream legs extend vertically to a height of some 4 or 5 ft. above the walls of the canal, and are left open at the top to provide a means of escape for any air which may be entrapped in the tubes below. (See Fig. 6.)

The pipes just mentioned are joined in such a way that water can enter them only through the perforations. By another $\frac{3}{4}$-in. pipe, attached to the base, the water entering the perforations is carried through the bulkhead, as shown in Fig. 6, and down stream for more than 100 ft. Here the pipe extends northward through the bottom of the canal wall into the gauge-house, where it is attached by a short piece of rubber hose to the standard tube (glass) 1 in. in diameter. The surfaces of the column of water in the tube and of the water in the chamber above the weir, are supposed to be in the same horizontal plane.

The south branch and the middle branch of this piezometer are shown clearly in Fig. 6. The former is 23 in. from the canal wall, the latter is 6 ft. farther north, and the branch on the north side is 22 in. from the north wall. The distance from the crest of the weir to the nearest perforation in the south branch of the piezometer is 25.67 ft.; to that in the north branch, 25.63 ft. The lengths of the perforated portions of the three horizontal pipes, beginning with the pipe on the south side, are 1.01, 0.92, and 0.92 ft., respectively; thus the average distance of the centers of the perforated portions of the pipes above the crest of the weir is 26.12 ft. The north branch is 6 ft. 2 in. above the bottom of the canal, or 5 ft. 1 in. below the horizontal plane containing the crest of the weir; the corresponding measurements for the south branch are 6 ft. 3 in. and 5 ft. 0 in., respectively.

The heads, as measured by this standard tube-piezometer, are used as a standard with which the heads measured by the other devices are compared. (See Plates XXXIII and XXXIV.)

(2).—With the 6-Ft. Longitudinal-Piezometer.

The 6-ft. longitudinal-piezometer consists of the following: A $\frac{3}{4}$-in. horizontal pipe perforated with four rows of holes 90° apart, there being thirteen $\frac{5}{32}$-in. holes in each row, with a longitudinal distance of 1 in. between their centers. Lines joining the diametrically opposite rows of holes make angles of about 45° with the vertical. The center line of this pipe is approximately $\frac{1}{2}$ in. from the south wall of the canal, and 3.8 in. below the horizontal plane containing the crest of the weir. The centers of the lines of holes in the pipe are 5 ft. $10\frac{1}{2}$ in. up stream from the crest. Eastward, the pipe extends 25 in. from the hole farthest east, and at this point the end of the pipe is closed with a

cap. Westward, the pipe extends through the bulkhead to the gauge on the bank of the canal below the weir, where the readings of this device are taken.

The pipe of the 11-ft. longitudinal-piezometer, which replaced the one just described and is similar to it, is the horizontal pipe shown in Fig. 6 attached to the canal wall. This pipe extends up stream from the crest of the weir, but is slightly below the level of the crest.

As shown on Plate XXXIII, the difference between the heads measured by the 6-ft. longitudinal-piezometer and those measured by the standard is almost constant, and is equal to 0.0032 ft., the standard tube-piezometer giving the smaller heads.

(3).—With the 6-Ft. Wall-Piezometer.

The 6-ft. wall-piezometer consists of a vertical $\frac{3}{4}$-in. pipe fixed or cemented into the canal wall in such a way that its perforated portion, containing $\frac{3}{16}$-in. holes, 6 in. apart, is flush with the plane side of the channel. The center of this pipe is 6.01 ft. up stream from the crest of the weir, and its uppermost hole is $28\frac{1}{4}$ in. vertically above the crest. At the top of the pipe, $37\frac{7}{8}$ in. above the crest of the weir (Fig. 5), a $\frac{3}{8}$-in. pipe-vent leads into the canal and thence vertically upward 1 ft. or more. At the bottom of the perforated pipe, which is about 12 in. below the crest of the weir, a connection projects into the channel and is attached to a pipe which extends westward through the bulkhead to the gauge. This piezometer gives slightly higher readings at all points than does the standard tube, and the difference between these readings is a uniformly increasing quantity from the lowest to the highest head measured, as shown on Plate XXXIII.

(4).—With the 6-Ft. Tape.

This tape was 2 ft. from the south wall of the canal and 6 ft. up stream from the crest of the standard weir.

For heads of less than 0.66 ft., the readings of the tape exceed those of the standard tube-piezometer, and for heads greater than this, the reverse is the case. (See Plate XXXIII.)

In the Cornell Hydraulic Laboratory Records* it is stated: "Notice that the standard tape [meaning what is here called the 6-ft. tape] is below the beginning of surface slope." This is the only indication on the laboratory records that the readings of this tape may

* Book 50. June 6th, 1901.

not be strictly relied on. If the measurements of this tape are affected by surface curvature, certainly all other measurements taken at the same distance up stream from the crest of the weir will be affected in a similar way. It is interesting to note, however, that, for all heads, both the 6-ft. longitudinal-piezometer and the 6-ft. wall-piezometer give readings greater than those measured by the standard tube-piezometer.

(5).—With the 10-Ft. Tape.

The 10-ft. tape was used in the center of the channel of approach 10.23 ft. up stream from the crest of the weir.

It was found necessary to use two right lines to show the relation existing between the heads as measured by this tape and the same heads as measured by the standard-tube piezometer. This relation is shown by a curve on Plate XXXIII.

(6).—With the 11-Ft. Wall-Piezometer.

The 11-ft. wall-piezometer consists of a ¼-in. opening in the canal wall, which leads into a vertical pipe extending upward inside a timber of the canal wall to the atmosphere, and downward a short distance below the opening; then it projects into the canal and is connected to a pipe which runs west along the south side of the channel and through the bulkhead to the gauge. The center of the opening is 10.70 ft. up stream from the crest of the weir and 0.3 ft. vertically below it.

One line, as shown on Plate XXXIII, represents the curve resulting from plotting the readings of this piezometer. It appears that for low heads the readings of the 11-ft. wall-piezometer are lower than those of the standard tube. This curve, however, would fit the points a little more accurately, as they are plotted, if, from its point of intersection with the standard line, it were drawn toward the origin. The difference, then, of the readings of this piezometer and those of the standard are zero for low heads, but they increase gradually until, for a head of 3 ft. by the standard-tube, the 11-ft. wall-piezometer gives a reading of 3.0398 ft. This difference, as seen on Plate XXXIII, makes a change of about 2% in the computed discharge of the weir.

(7).—With the 2-Ft. Wall-Piezometer.

The perforations of the 2-ft. wall-piezometer, made to receive the water, are in a vertical ¾-in. pipe, buried or cemented in the wall of

the canal in such a way that the portion containing the perforations forms a part of the plane side of the channel. The top of this pipe, which is 37⅝ in. vertically above the horizontal plane containing the crest of the weir, has attached to it a ⅜-in. pipe-vent leading into the canal and then vertically upward a distance of about 1 ft., as shown in Fig. 5. At the bottom of the perforated pipe, some 12 in. below the horizontal plane containing the crest of the weir, a connection leads into the channel and thence westward, close to the canal wall, through the bulkhead to the gauge on the bank of the canal.

The horizontal distance from the crest of the weir to the vertical line containing the centers of these perforations is 24½ in. The center of the lowest hole is 5⅛ in. below the horizontal plane containing the crest. From this point vertically upward, there are twenty $\frac{3}{16}$-in. holes 2 in. apart. The curve which results from plotting the heads as indicated by this piezometer is shown on Plate XXXIII. As only part of the curve could appear on the plate drawn to the large vertical scale used in the upper part of the diagram, the whole curve is drawn to a smaller scale in the lower portion.

(8).—With the Crest-Piezometer.

This piezometer consisted of a ½-in. brass pipe, about 1 ft. long, set vertically in an upright plank which was buried in the concrete in such a way that the plank and the perforated portion of the pipe formed a part of the plane side of the canal. The centers of the perforations (⅛ in. in diameter and 1 in. apart) were vertically above the crest of the weir at its south end (Fig. 5). The lower end of this tube was attached to a pipe which led to the top of the bank of the lower canal where the gauge was placed.

The curve determined by the readings of this piezometer is shown on Plate XXXIII in the lower diagram, which is drawn on the smaller vertical scale. The great difference between the readings of this piezometer and those of the point-gauge at the crest is one of the most surprising and interesting features of this comparison.

(9).—With the Point-Gauge at the Crest (Bent).

The operation of this gauge is much the same as that of the Boston level rod when the line of sight is below the horizontal plane containing the bench-mark. The target, however, is replaced by a ¼-in., straight, round, pointed bar, extending about 6 in. below the end of

the sliding portion of the rod. Instead, too, of having the bottom of the stationary part of the rod rest on the reference point or bench-mark, a small brass bracket is fastened to the back of this portion of the rod and is used to support it. The two ends of the rod are placed in the rectangular notches already mentioned, and thus the point is kept in the same vertical line when it is moved in taking a measurement. (See page 1261.)

The readings which determine this particular curve (Plate XXXIII, lower portion) were taken, supposedly, in the vertical plane containing the crest of the weir, but the statement is made* that when the point was at the crest level it was $\frac{7}{16}$ in. down stream from this plane. It is for this reason that the curve is marked "bent."

(10).—With the Point-Gauge at the Crest.

(After the point was straightened.)

The readings taken with this gauge, after the point was straightened so that it moved in the vertical plane of the crest of the weir, have been used to determine another curve, shown on the lower portion of Plate XXXIII. The description of this gauge is given on pages 1261 and 1267.

C.—Measurements of Heads at the Standard Weir (Height 6.65 Ft.).

(1).—With the 6-Ft. Francis Piezometer.

This device for measuring the depth of water on the crest of the weir consists of a ¼-in. circular orifice in a 6 by 12-in. brass plate; the center of the opening is 6 ft. 0⅛ in. up stream from the crest of the weir, 4⅞ in. below the upper edge of the plate, and 2¼ in. below the horizontal plane containing the crest of the weir. This orifice leads into a pipe which extends through a timber of the canal wall and is attached to the gauge on which the "Francis" readings were taken.

This instrument, like all the others from which readings were taken at this weir (the curves being shown on Plate XXXIV), gives readings or heads smaller than the standard-tube piezometer with which it is compared. This, however, is to be expected, even if the readings are taken simultaneously, because the standard-tube measurements were taken at the weir 11.25 ft. high and the others at the weir having a height of 6.65 ft. The vertical distance between these curves, however,

* Book 50, Cornell Hydraulic Laboratory Records, May 24th, 1901.

is the same as if the comparison had been made with the heads of any other of the measuring devices.

(2).—With the 6-Ft. Bottom-Piezometer.

The orifice of this measuring device is a ¼-in. circular opening in a ¾-in. cap which is flush with the face of an 8 by 10-in. timber forming a part of the side-wall of the canal. The opening is 1 ft. above the bottom of the channel of approach and 6 ft. up stream from the crest of the weir. The pipe to which the cap is attached extends through the canal wall, where it is connected to the gauge.

The curve resulting from plotting the readings of this piezometer is shown on Plate XXXIV.

(3).—With the 6-Ft. Float-Piezometer.

This piezometer consists of a plank, a ¾-in. pipe extending half way across the bottom of the canal, and a gauge on the outside of the canal. The 2 by 12-in. plank, 18 ft. long, and dressed on the north side, is on edge in the bottom of the canal with one end against the bulkhead; the other end is beveled at an angle of about 30° with the center line of the canal, for the purpose of directing away from the piezometer opening all water having a disturbed direction of flow. In the dressed face, which occupies the center of the canal, is placed a ¾-in. brass plug which contains a ¼-in. circular orifice leading into the ¾-in. pipe just mentioned. This pipe extends across the channel, through an 8 by 10-in. timber of the south wall, and is attached to the gauge just outside of the canal.

The center of the circular orifice which receives the water is 3 in. above the bottom of the canal and 6.00 ft. up stream from the crest of the weir.

The curve determined by plotting the readings of this piezometer is shown on Plate XXXIV.

(4).—With the 6-Ft. Point-Gauge.

This instrument is described on pages 1261 and 1267. Readings with it, which determine the curve shown on Plate XXXIV, were taken 6.00 ft. up stream. As this curve is determined by only sixteen points, the results it indicates have less weight than those given by curves determined by many more.

(5).—With the 10-Ft. Tape.

Readings by this tape were taken 10 ft. 4⅝ in. up stream. Its curve is shown on Plate XXXIV.

(6).—With the 13-Ft. Three-Tube-Piezometer.

This piezometer is constructed in the following manner: Three ¾-in. pipes (each 41½ in. above the bottom of the canal, and each perforated for 1 ft. of its length) are attached by 90° elbows to six legs, one at each end of each pipe; these legs are connected by T's to two transverse and three longitudinal pipes which form a base on the bottom of the canal. Through all these pipes the water circulates freely and conducts the pressure into a ¾-in. pipe, more than 100 ft. long, on the bottom of the channel, to an opening near the bottom of the canal wall, through which the pipe extends to the "lower tube" (in the gauge-house), on which the readings of this instrument were taken.

The three pipes containing the perforations are parallel to the sides and bottom of the canal. In each pipe there are four rows of $\frac{3}{16}$-in. circular holes. These rows are 90° apart, and the holes in the rows are 1 in. apart. The perforations farthest up stream are 31½ in. down stream from the up-stream legs previously mentioned, and the perforations continue down stream to within 6½ in. of the down-stream legs.

The curve which the readings of this piezometer determine is shown on Plate XXXIV.

(7).—With the 15-Ft. Tape.

Heads were measured with this tape 14.82 ft. up stream. The curve determined by plotting these heads is shown on Plate XXXIV.

(8).—With the 16-Ft. One-Tube-Piezometer.

The 16-ft. one-tube-piezometer had only one perforated pipe; the 13-ft. three-tube-piezometer had three. The former instrument was replaced by the latter,* and the readings of both these piezometers were taken on the "lower tube" in the gauge-house.

This single perforated tube was 42 in. above the bottom of the canal and parallel to the direction of the flow, the center of the line of perforations being 15 ft. 8¾ in. up stream and 8 ft. 7½ in. from the south side of the canal.

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* Cornell Hydraulic Laboratory Records, October 18th, 1901.

The results obtained by measuring the heads with this piezometer are shown by the curve for this instrument on Plate XXXIV.

(9).—With the Point-Gauge at the Crest.

The point-gauge has been described on pages 1261 and 1267. The differences between the readings of this instrument and those of the "standard" are so great that they cannot be shown on Plate XXXIV, therefore the equations of the curve are given instead of the curve itself.

Let y = the readings of the point-gauge.

and x = the reading of the "standard," all in double centimeters.

For values of x between 14 double centimeters and 39.1 double centimeters,

$$y = 0.8427\ x + 0.125.$$

For values of x between 39.1 double centimeters and 41.5 double centimeters,

$$y = 0.8016\ x + 1.734.$$

(10).—With the Hook-Gauge in the Bazin Pit.

With the intention of measuring the heads on the lower standard weir (height 6.65 ft.) exactly as Bazin measured those on his standard weir, and thus justifying the use of Bazin's formula for computing the discharges over this weir, the "Bazin pit" was constructed in 1903.

Bazin's standard weir was 1.135 m. high, and he measured the heads on this weir in a chamber by the side of the canal into which the water entered through a 4-in. circular opening, the end of which was flush with the canal wall. The center of this opening was 5 m. up stream from the vertical plane containing the crest of the weir, thus making the ratio of the distance up stream, where heads were measured, to the height of the weir, 5 ÷ 1.135, or 4.42.

The height of the Cornell weir is 6.65 ft. and the distance up stream to the center of the 4-in. opening (3¾ in. above the bottom of the canal) is 29.88 ft., making the ratio above mentioned 29.88 ÷ 6.65, or 4.49, a value almost exactly the same as that given by the conditions at Bazin's weir.

The Bazin pit, 5 ft. long, 3 ft. 4 in. wide, and 10 ft. 8 in. deep, has its length at right angles to the direction of the canal. The distance between the canal and the pit is 3 ft. 6 in. The sloping roof over this pit is shown at the left of Fig. 3.

The height of the water in the Bazin pit was measured with a hook-gauge which was firmly fastened to the timbering on the west side of the pit.

The relation between the heads measured in the Bazin pit and the same heads measured with the 15-ft. tape has been deduced from the readings herewith presented, these readings having been taken simultaneously. (See Table B.)

Letting y = heads by the 15-ft. tape, in feet;

and x = heads by Bazin pit, in double centimeters;

the equation.

$$y = 0.06535\ x + 0.0029,$$

expresses the relation between them for heads up to 4 ft. From the relation thus expressed, the curve for the Bazin pit on Plate XXXIV has been drawn.

III.—Hydraulic Laboratory of Cornell University.

A.—Description.

At the north boundary line of the campus of Cornell University, and on the south bank of the magnificent Fall Creek Gorge, is located Cornell's famous Hydraulic Laboratory. Fig. 1 shows the lower standard weir (height 6.65 ft.), a part of Fall Creek Gorge, the falls, the dam, the laboratory building, Beebe Lake, and the west end of the canal, as they appear during the high-water season. Fig. 2 is a clearer view of the laboratory building, the gorge, and the dam. Fig. 3 shows the west or lower end of the canal. The standard weir (height 11.25 ft.) at the east or upper end of the canal, Beebe Lake, and a portion of the forest on the campus of Cornell University are shown in Fig. 4. The canal is 10 ft. deep, 16 ft. wide, and 350 ft. long between the two standard weirs; its walls are of smooth concrete, and 18 in. thick.

The upper weir chamber, at the east end of the canal, is 59 ft. long, 17.7 ft. deep, and 16 ft. wide. It contains a baffle, 46 ft. above the weir. There are six sluice-gates at the head or east end of the canal, and the 10-ft. and 16-ft. tapes are located there. The position of the baffle below this weir can be seen in Fig. 4. The six sluice-gates are operated with rack and pinion by iron levers about 6 ft. long.

As detailed drawings and descriptions of the Hydraulic Laboratory

of Cornell University have appeared in several publications,* no additional matter need be given here. The various gauges, tapes, piezometers, and other devices used for measuring heads have already been described.

B.—Method Used for Measuring Water.

In the Cornell Laboratory there was no equipment by which the discharge could be measured volumetrically. The quantity of water which flowed over the standard weirs for known heads was compiled by using the weir formulas of Francis, Fteley and Stearns, and Bazin.

Experiments were made by running the same quantity of water over the two weirs in succession, measuring the heads on both, and from these heads, by use of the three weir formulas named, the discharge was computed.

For computing the quantities of discharge in this paper, Bazin's formula has been used, or, what is equivalent to the use of this formula, the curve previously mentioned (page 1242), which was constructed for the standard weir of height 6.65 ft.

The Bazin pit (page 1271) was put into actual use for the first time during the afternoon of June 22d, 1903. All heads read in this pit are used directly on the diagram for ascertaining the corresponding discharge.

By a long series of comparisons, the relation between the heads measured in the Bazin pit and those measured by some other method simultaneously, have been found, and this has been expressed in the form of equations. By the use of these equations, all heads read on either of the standard weirs have been reduced to equivalent Bazin pit readings before being used in the diagram.

It may be said, therefore, that the quantities of water herein given as being measured in the Cornell Hydraulic Laboratory have been determined by using a weir similar to Bazin's standard. The heads on this weir were measured just as Bazin measured them, and the quantities were computed from the heads thus read by using Bazin's formula (page 1242).

* *Engineering News*, Vol. 41, March 2d, 1899. *Transactions*, Am. Soc. C. E., Vol. XLIV, p. 285. "Report on New York Water Supply," by John R. Freeman, M. Am. Soc. C. E., p. 129. "Accuracy of Stream Measurements," by E. C. Murphy, M. Am. Soc. C. E., for U. S. Geological Survey, p. 60. "Weir Experiments, Coefficients, and Formulas," by Robert E. Horton, M. Am. Soc. C. E., Water Supply Paper No. 150, U. S. Geological Survey, p. 86.

In all the Cornell experiments a correction has been applied for leakage. Leakage runs were made frequently in order to determine the amount of this correction. By using saw-dust to stop up the small openings around the gates in the lower end of the canal, the leakage was kept small.

C.—Method of Running Experiments on Models.

Some of the models on which experiments were performed were built over the upper standard weir (height 11.25 ft.) and others were built over the lower standard weir (height 6.65 ft.). When a model was built over one weir, the quantity of water used during the experiments was measured over the other. After opening the sluice-gates at the upper end of the canal a sufficient height to give approximately the head desired, and waiting from 10 to 30 min. for conditions to settle, readings on the various devices for measuring the head on the model and on the standard weir were begun, and these were continued for from 10 to 30 min. The means of the readings taken during a run were used to determine the head for that run or experiment. The readings for all the devices from which the mean readings were derived were taken during the same period of time.

The differences between these mean readings and the zeros of the instruments used were the head required (page 1261).

While the experiment or run was in progress, independent check readings were taken on all the measuring devices.

IV.—Hydraulic Laboratory of the University of Utah.

A.—Location.

The Hydraulic Laboratory of the University of Utah is on the small city reserve, in the eastern part of Salt Lake City, just north of the University campus. This enclosure contains the Thirteenth East Street Reservoir (a part of the city water-works system), the Hydraulic Laboratory of the University of Utah, and a small area surrounding them. Figs. 7, 8, and 9 are general views of the laboratory and the reservoir. Fig. 13 contains a general plan and two cross-sections of these structures. The authorities of Salt Lake City have very kindly permitted the University to construct the canals, pipe lines, measuring basin, and laboratory building on city property, and

allow the city water coming to this reservoir to be used in the laboratory.

B.—Source of Water Supply.

The water supply for the Thirteenth East Street Reservoir comes from Emigration, Parley's, and Big Cottonwood Canyons, which are in the Wasatch Mountains, south and east of Salt Lake City.

C.—Pipe Lines.

The water is carried to a settling tank and valve-house east of the reservoir in closed pipes and conduits. From this valve-house the two pipes leading directly west are shown in part on Fig. 13. One is an 18-in., vitrified pipe, the other a 12-in. cast-iron pipe. The 18-in. vitrified line leads into the reinforced concrete "intake" box of the laboratory, from which water is taken to the laboratory proper through an 18-in. machine-banded wood-stave pipe, 750 ft. long. The 18-in. vitrified line leading from the "intake" to the reservoir serves only as an overflow, as its upper end is some 4 ft. higher than the upper end of the pipe leading to the laboratory. This overflow pipe has at its upper end a 90° bend into which, bell-end up, a length of vitrified pipe is fastened. This gives the outflow line a comparatively great carrying capacity for only a slight increase of head; thus it assists greatly in keeping constant both the heads and discharges in the laboratory. The valves regulating the quantity of water used in the laboratory are at or near the lower end of the supply pipe. As the fall from the intake box to the laboratory is about 70 ft., the hydrostatic pressure available in the laboratory from this source is about 30 lb. per sq. in. There is available for use in the laboratory, however, a supply of water from a 6-in. city main which will produce a hydrostatic pressure of about 160 lb. per sq. in. When the equipment for the building is put in place, a connection will be made with this line.

D.—Canals and Measuring Basin.

The canals and measuring basin in the laboratory are of concrete reinforced with steel rods. The walls are vertical on the inside and battered on the outside. The floors or bottoms of these structures are practically horizontal. It would be much easier to clean them if the bottoms had some slope.

(1).—Canal No. 1.—Baffle.

At the upper end of Canal No. 1 (shown at the right of Fig. 10) there is a receiving basin 8 ft. square having on three sides walls 10.28 ft. high. The water is discharged vertically from the wood-stave pipe into this basin through an 18-in. cast-iron bend, which is shown in the cross-section of the south end of the laboratory building.

From this basin the water flows through a baffle (marked on Fig. 13) into Canal No. 1. The bottom of this canal is 1 ft. higher than that of the intake basin. Its walls are 6.28 ft. high, and it is 2 m., or 6.56 ft., wide and 50 ft. long. This canal is seen at the right of Fig. 10.

This baffle is near the upper end of the canal, and is of timbers, 6 in. wide and 1½ in. deep, which fit into vertical grooves in the canal walls (Fig. 10). The individual timbers are 6 in. wide on the up-stream side, 3 in. wide on the down-stream side, and 6 in. thick horizontally. One of these timbers can be seen above the water in Fig. 10. The space between these timbers is 3 in. wide on the up-stream side, 6 in. wide on the down-stream side, and 6 in. across in a horizontal direction, the same as the timbers. These timbers are fairly well shown in Fig. 11, which is a view of the baffle in the upper end of the measuring basin. These three baffles are similarly constructed, and their locations are marked on Fig. 13. The theory on which these baffles are built is that both the "head" and the velocity of the water passing through these spaces, or rather "sudden enlargements," will be greatly decreased.

(2).—Canal No. 2.

From Canal No. 1, water can flow into Canal No. 2 in two ways—either over the weir in the lower end of the former channel or through the 12-in. by-pass valve just outside the canal near its lower end. From the bottom of Canal No. 1 to the bottom of Canal No. 2 there is a vertical drop of 5 ft. This canal, like the former, is 2 m., or 6.56 ft., wide and 50 ft. long. Its walls are 8.28 ft. high. The baffle is similar to that just described. The upper timber of the baffle, and the whole of this canal, with the weir and by-pass valve at its lower end, are shown clearly in Fig. 10.

(3).—Canal No. 3.

From Canal No. 2 water can flow into Canal No. 3 in the same two ways as those just described, namely, over the weir and through

the 12-in. by-pass valve (Fig. 10). The straight portion of this canal is 62 ft. long, the walls are 3 ft. high, and its width is also 6.56 ft. Its bottom is at the same elevation as that of the one above it. Instead of using a valve for a by-pass from this canal, a hole, semicircular at the top and 6 in. in diameter, with a width of 6 in. at the bottom, is cut through the wooden bulkhead at the bottom of the canal. A piece of 1-in. lumber covered with rubber packing or gasket serves to keep water from flowing through this opening.

(4).—The Measuring Basin.—Method of Determining Capacity.

The measuring basin in this laboratory is 150 ft. long, 16 ft. wide, and its walls are 10 ft. high. Therefore its nominal capacity is 24 000 cu. ft., or 180 000 gal. The west wall is shown just above the coping of the reservoir wall in Fig. 7. The inside of the basin can be seen in Fig. 8, also the bulkhead of 12 by 12-in. timbers at its lower end, with the stem of the 12-in. by-pass valve just above and to the right of them. The top of the baffle in the upper end of the basin is shown only slightly in Fig. 8. It is seen clearly, however, in the lower portion of Fig. 11. A good view of the west wall of the basin with its buttresses is shown also in Fig. 12. The wasteway at the lower end of the basin, with water flowing over it, is shown at the left of Fig. 7.

At the upper end of the basin there is a cast-iron waste-gate, 2 ft. wide and 2 ft. 6 in. long, which rotates on hinges at its upper edge. When closed, the plane of this gate makes an angle of 30° with the vertical; when open, it is practically horizontal. The gate is opened with a chain, one end of which is attached to the gate and the other to a round 2-in. rod operated with a lever and ratchet.

The water from this gate and that wasted by the diverting apparatus flows into the reservoir through the reinforced concrete channel shown in Figs. 7, 9, and 12.

By careful measurements, the areas of horizontal cross-sections of this basin, 1 ft. apart, have been ascertained. The lengths and widths of these sections were found by using two plumb-bobs. For example, to find the length of a particular section, the two plumb-bobs were hung near the ends of the section and in a plane parallel to the sides of the channel. The distance required is that between the two strings, plus the two distances of the strings from their respective walls. The distance between the strings was measured accurately and conveniently

with a steel tape, and that from the wall to the string, at any desired point, with a rule graduated to hundredths of a foot.

(5).—Diverting Device.

As shown in Fig. 12, the device for diverting the water into and over the edge of the measuring basin is a car, or sloping channel, on wheels. With the car in the position shown, the water is flowing into the measuring basin. If the car were pushed to the right until its vertical ends and those of the walls of the canal were in contact, the water would flow along the bottom of the car and be discharged outside of the wall of the measuring basin and into the concrete wasteway shown in the figure. The plan and cross-section of this portion of the laboratory (Fig. 13) also show how this device operates. The inside of this car, in horizontal projection, measures 8 ft. in the direction of the slope and 7 ft. at right angles to it, and the bottom has a slope of 1 ft. 9 in. The wheels run on iron rails. The channel or body is lined with galvanized iron. At the east end of the car (Fig. 12) there is a framework the vertical members of which are channel irons in which a wooden gate slides. The gate can be lowered suddenly with the trip shown in the figure. If the gate is lowered in this way, when a comparatively large stream of water is being wasted, the force of the water assists materially in pushing the car to the left so as to allow the water to flow into the measuring basin. For small streams, the sharp and turned-up cutting edge of the galvanized-iron lining of the bottom of the car, seen clearly in the figure, can be passed easily and quickly through the flowing stream without the aid of the water. The time at which the diversion is made can be observed accurately, and therefore the time of beginning and closing a run can be determined.

(6).—Weirs.

At the lower ends of the concrete canals there are weirs with sharp edges and without end contractions. These are of 4 by 4-in. timbers with 6 by 3½ by ⅜-in. angle irons on top, the 6-in. leg being vertical. The edge of this vertical leg is machined carefully and placed accurately in a horizontal position.

The crest of the weir in Canal No. 1 is 3.72 ft. above the bottom of the channel of approach; that is, its height is the same as that of

Bazin's standard weir. The height of the weir in Canal No. 2 is 6.65 ft., the same as that of the lower standard weir in the Hydraulic Laboratory of Cornell University; the height of Weir No. 3 is 1.64 ft. The nominal lengths of all these weirs is 2 m., or 6.56 ft.

Weir No. 1, for all the experiments made on it and contained herein, had its length cut down from 6.56 ft. to 1.77 ft., so as to get a higher head with the water available. One side of the channel of approach for this short weir was composed of a framework of 2-in. plank which extended 20 ft. up stream. Water stood at the same elevation on both sides of this partition. The bulkhead constructed in the vertical plane of the weir was made water-tight.

(7).—Pits for Stilling Water.

Above Weirs Nos. 1 and 2, pits have been built on each side of and touching the outside of the canal. These have been patterned after those used by Bazin.

Above each of these two weirs there are four pits, shown on Fig. 13; two are 6 ft., and two are 16 ft. 6 in., up stream from the crest. These pits have the same depth as the canal to which they are attached. The water is admitted to them through a 4-in. pipe having its ends cut off flush with the side of the channel. These pits have equal horizontal measurements of 1 ft. 6 in. inside.

E.—Measuring Heads.

Although the actual heads on these weirs could be measured most accurately, perhaps, in the stilling pits just described, they have not been measured in this way because, under usual conditions in practice, this cannot be done. As the reading of heads with a tape and plummet is a method which can be applied easily to almost any case, it has been adopted in the Hydraulic Laboratory of the University of Utah.

For measuring heads on Weirs Nos. 1 and 2, each tape is 15 ft. up stream from the weir crest. For Weir No. 3 the tape is 5 ft. up stream from the crest. The zero readings of these tapes were determined in the manner described on page 1261.

The depths of water in the measuring basin at the beginning and end of a run are measured by two tapes simultaneously, one near the south and the other near the north end of the measuring basin.

V.—Notation.

Q = Quantity of flow, in cubic feet per second per foot of length of weir;

Q_m = Quantity of flow, in cubic feet per second per foot of length of weir, as actually measured;

Q_c = Quantity of flow, in cubic feet per second per foot of length of weir, as computed by the trial formula, log. $Q_c = 1.5000 \log. h + 0.5000$;

Q_B = Quantity of flow, in cubic feet per second per foot of length of weir, as computed by the Bazin formula;

h = Head, in feet, on the weir, or the vertical distance between the plane containing the surface of the water in the channel of approach and the horizontal plane containing the crest of the weir;

Ht. or Height = Height, in feet, of the crest of the weir above the bottom of the channel of approach;

"15-Ft. Tape Head" means that the head on the weir was measured with a tape and plummet 15 ft. up stream from the crest of the weir;

K = the term $\left[1 + 0.55 \left(\frac{h}{p+h}\right)^2\right]$, in the Bazin formula:

$$Q_B = \frac{2}{3}\left(0.6075 + \frac{0.0148}{h}\right)\left[1 + 0.55\left(\frac{h}{p+h}\right)^2\right] h \sqrt{2gh}.$$

VI.—Comments and Acknowledgments.

Much of the information contained in this paper, as has, no doubt, been observed, is based on the results of investigations carried on in the Hydraulic Laboratory of Cornell University. These experiments were all performed under the close personal supervision of Mr. Williams, to whom the writer is indebted for several corrections in the descriptions of apparatus used, and for many helpful suggestions during the whole period covered by the preparation of this paper. It was Mr. Williams who pointed out the way that led to this study, and who, while the writer was a graduate student at Cornell University, drew the writer's attention to the fact that "the formula, $Q = m\,h^n$, is a perfectly logical one, and about the simplest that can be applied to the flow of water."*

* *Transactions*, Am. Soc. C. E., Vol. XLVII, p. 369.

The methods used in working up the data are largely original, therefore, the progress made has been slow. Were the work to be done again, it could be accomplished in a fraction of the time that has been spent on it.

To Dr. Schoder, at present Engineer in Charge of the Hydraulic Laboratory of Cornell University, to Mr. T. J. Rodhouse, scholar in Civil Engineering, to Mr. Albin H. Beyer, to Claude Berry, and Charles R. Wyckoff, Associate Members, Am. Soc. C. E., and to Mr. K. B. Turner, instructors and graduate students doing experimental work in hydraulics in Cornell University during the years 1902-04, the writer is indebted for assistance in performing the experiments, and in making computations on the original data.

For assistance in conducting the experiments in the Hydraulic Laboratory of the University of Utah and in making the final computations, the writer is indebted to Mr. Frank W. Becraft, and for the excellent work in drafting shown on the plates he is indebted to Mr. Hugh C. Lewis, University of Utah. The writer must also mention the fact that valuable suggestions and assistance have been received from Messrs. Thomas F. McDonald, A. Z. Richards, T. Ross Wilson, Jr., and Junius J. Hayes, all graduates of the State School of Mines, University of Utah, and former students in hydraulics in the classes conducted by the writer.

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DISCUSSION

Mr. Moritz.

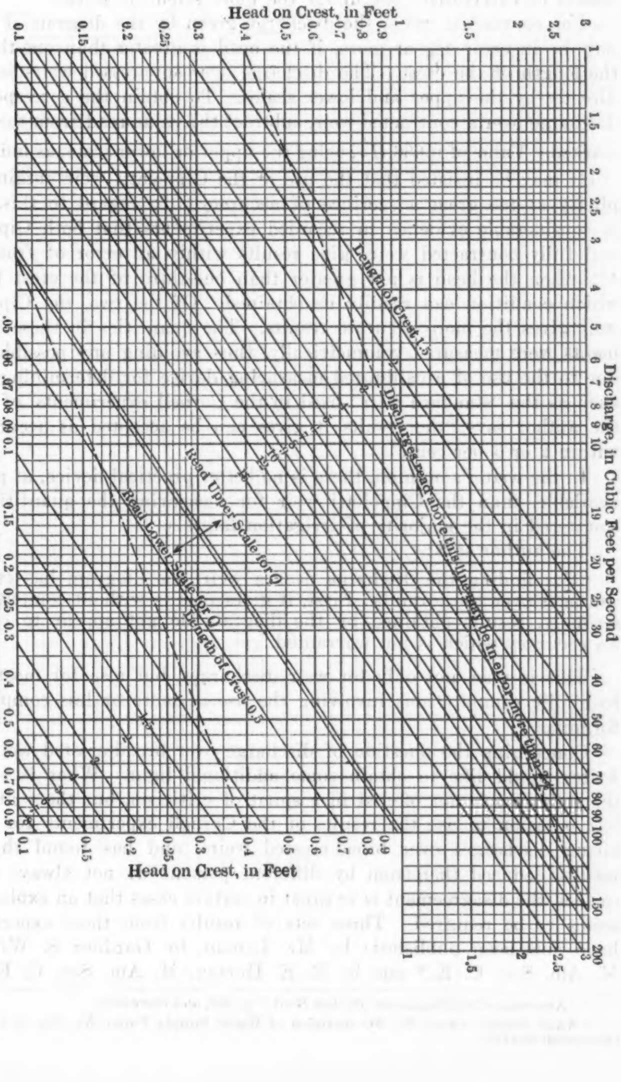
E. A. Moritz, Assoc. M. Am. Soc. C. E. (by letter).—In making analyses of all the data and preparing this paper the author has performed a monumental task which, no doubt, will receive the cordial appreciation of all hydraulic engineers.

The author states that "the method of calculating the discharge over a weir should be the simplest that can be found; in other words, it should give the most accurate results with the smallest amount of work", and he advocates the sharp-crested weir without end contractions as most nearly fulfilling these requirements. The writer agrees with the statement in so far as the accuracy of the results is concerned, provided the height of the weir crest is properly determined each time a measurement is made or the approach channel is kept free of silt; but either of these requirements necessitates considerable care and effort, so that the author's statement is open to serious question. The writer has in mind an irrigation project, typical of many, on which the use of the sharp-crested weir without end contractions would be practically impossible on account of the large quantities of silt carried at certain seasons of the year, which would necessitate either cleaning out the approach channel at frequent intervals or determining the height of the crest at each measurement. Either alternative is impracticable, as any one knows who is familiar with irrigation projects having from 1 000 to 2 000 or more measuring weirs. This example is selected because the author evidently had in mind the use of the weir which he advocates for the measurement of irrigation water, for he says: "the sharp-crested weir without end contractions can certainly be used to best advantage in all irrigation projects".

For this purpose the Cippoletti weir offers a simple, convenient, and accurate method, and is used probably more than any other type. The construction of the Cippoletti weir is fully as simple as the suppressed weir, if not more so, notwithstanding the author's statements in this regard. In most cases a pool of sufficient size can be excavated above the weir to avoid any but negligible velocities of approach, and one or two good cleanings of the pool during a season will generally suffice. For this weir, tables or diagrams can be prepared which will give at a glance the discharge for any measured head, or, as is frequently necessary, the head can be read which will give any required discharge. Fig. 46 represents such a diagram for Cippoletti weirs, and Fig. 47* is for contracted and suppressed weirs with vertical sides (Francis' formula). The importance of simplicity in this regard cannot be exaggerated, on account of practical considerations. The

* Francis formula for contracted rectangular weirs, $Q = 3.33\ H^{\frac{3}{2}}\ (L - 0.2\ H)$, may be written $Q = 3.33\ L\ H^{\frac{3}{2}} - 0.666\ H^{\frac{5}{2}}$, or $Q =$ (Discharge over suppressed weir) $- 0.666\ H^{\frac{5}{2}}$

Mr. Moritz.



DISCHARGE OF STANDARD CIPPOLETTI WEIRS $Q=3.37\ L\ H^{3/2}$

FIG. 46.

Mr. Moritz. measurements and care required for the accurate use of the author's diagrams vitiate to a large extent their practicability, although they should be exceedingly useful for the more scientific works.

For contracted weirs, the discharge given by the diagram, Fig. 47, may be in error 4% or more, if the head is greater than one-third of the length of the crest. The discharge over suppressed weirs is given directly by the upper and lower scales. To obtain the corresponding discharge over a contracted weir, subtract the value read from the curve marked "Value of $0.666\, H^{\frac{5}{2}}$, using the upper or lower scale, as indicated.

It may be insisted that the use of the Cippoletti weir obtains simplicity at too great a sacrifice of accuracy. In regard to this, there is considerable evidence in recorded experiments that both Cippoletti and fully contracted weirs give results within an error of from 2 to 4% when the head is not greater than one-third of the crest length, which condition can readily be obtained. Of the two, the Cippoletti weir gives the more accurate results. The quantities in these experiments were measured volumetrically, thus avoiding any possible error due to the use of a calibrated measuring device for determining them, such as the "standard weir" used in the Cornell experiments, of which the author says "the results it gives can be accepted as accurate to within 2 or 3 per cent."

In the writer's opinion, there is no better practical device, at present available, than the Cippoletti weir for measuring the quantities encountered in the majority of irrigation laterals.

The author says:

"For finding the discharge over a weir, the diagram has two advantages over the formula: First, it gives results without computation; secondly, results obtained by the diagram do not appear to contain an accuracy which is not warranted."

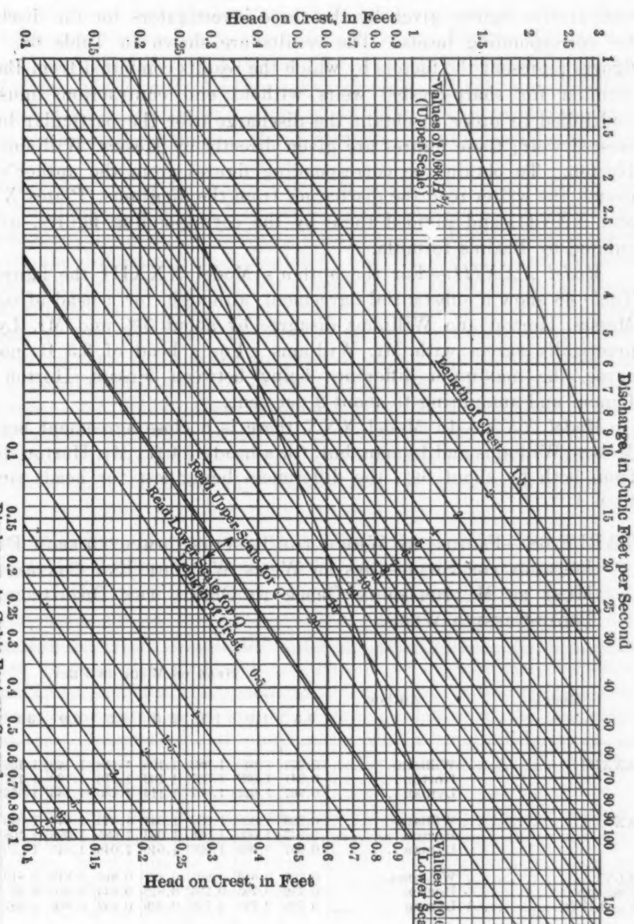
This is true, not only for weir discharges, but also for most other hydraulic computations involving the use of more or less complicated formulas.

Concerning the question of discharge over broad-crested weirs, the writer would like to obtain some additional light. What is said in the following is not offered in a spirit of criticism, but the writer has had occasion to use the results of the Cornell experiments for calculating discharges over broad-crested weirs, and has found that the results deduced therefrom by different persons do not always agree; in fact, the disagreement is so great in certain cases that an explanation seems to be required. Three sets of results from these experiments have now been published: by Mr. Lyman, by Gardner S. Williams, M. Am. Soc. C. E.,* and by R. E. Horton, M. Am. Soc. C. E.†

* "American Civil Engineers' Pocket Book," p. 869, and elsewhere.

† Water Supply Paper, No. 200 (revision of Water Supply Paper, No. 150) of the U. S. Geological Survey.

Mr. Moritz.



DISCHARGE OF STANDARD SUPPRESSED RECTANGULAR WEIRS: $Q = 3.33\,LH^{3/2}$

DISCHARGE OF STANDARD CONTRACTED RECTANGULAR WEIRS: $Q = 3.33\,LH^{3/2} - 0.666\,H^{5/2}$

FIG. 47.

Mr. Moritz.

The writer has selected at random several types of the broad-crested weir on which experiments were made, and has set down the comparative figures given by the three investigators for the discharge for corresponding heads. The results are shown in Table 68. The figures represent the factors by which the results computed from Bazin's formula for sharp-crested weirs without end contractions must be multiplied in order to obtain the discharge over the particular broad-crested weir; these figures are given directly by Messrs. Williams and Horton. To obtain the corresponding figures from the author's diagrams, the writer read the discharges from the diagrams (Plates XXXI and XXVI), and divided these by the corresponding figures, as calculated by Bazin's formula.

Model XXXIII.—For the author's Model XXXIII the figures in Table 68 show a substantial agreement, although, for a head of 0.5 ft., Messrs. Horton and Williams disagree by about 3% and Mr. Lyman practically agrees with Mr. Williams. For a head of 3.5 ft. no two agree, the maximum difference being between Messrs. Horton and Lyman and amounting to about 2 per cent.

Model XXX.—In Model XXX there is a close agreement between Messrs. Williams and Lyman, but, for a head of 4 ft., Mr. Horton differs from both by about 6%, the differences being less for heads smaller than 4 ft.

TABLE 68.—FACTORS BY WHICH THE RESULTS COMPUTED FROM BAZIN'S FORMULA FOR SHARP-CRESTED WEIRS WITHOUT END CONTRACTIONS MUST BE MULTIPLIED IN ORDER TO OBTAIN THE DISCHARGE OVER BROAD-CRESTED WEIRS.

Model.	Author.	Head on Weir, in Feet.							
		0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
XXXIII	Williams	0.971	1.040	1.083	1.105	1.118	1.128	1.136	1.144
	Horton	0.941	1.039	1.087	1.109	1.118	1.120	1.127	1.123
	Lyman	0.967	1.032	1.067	1.114	1.130	1.143	1.152	1.140
XXX	Williams	0.968	1.008	1.032	1.041	1.043	1.044	1.045	1.046
	Horton	0.947	1.000	1.036	1.063	1.085	1.096	1.108	1.110
	Lyman	0.967	0.985	1.030	1.040	1.048	1.045	1.050	1.043
XLVI, Flat top, $b = 3.17$ ft.	Williams	0.797	0.812	0.821	0.821	0.816	0.813	0.810	0.808
	Horton	0.792	0.795	0.796	0.815	0.844	0.870	0.90	0.93
	Lyman	0.785	0.795	0.792	0.823	0.852	0.866	0.887	0.898
XLIII and XLIII*a*, Flat top, $b = 16.29$ ft.	Williams	0.783	0.792	0.797	0.795	0.789	0.784	0.780	0.777
	Horton	0.790	0.790	0.792	0.793	0.793	0.791	0.791	0.789
	Lyman	0.785	0.803	0.801	0.792	0.780	0.786	0.792	0.788
XLI, Flat top, $b = 1.65$ ft.	Williams	0.819	0.879	0.910	0.925	0.932	0.938	0.942	0.947
	Horton	0.806	0.808	0.878	0.906	0.985	1.00	1.00	1.00
	Lyman	0.785	0.825	0.891	0.933	0.965	1.00	1.009	0.995

Model XLVI.—The greatest variation is found in Model XLVI, having a flat-top crest 3.17 ft. wide, where the difference between Messrs. Williams and Horton for a 4-ft. head amounts to about 13%, the differences decreasing as the head decreases. Messrs. Horton and Lyman agree much better, but they also show quite large differences. Mr. Moritz.

Models XLIII and XLIIIa.—For Models XLIII and XLIII*a*, having a flat-top crest with a width of 16.29 ft., all three investigators are in substantial agreement, the differences for any head being very small.

Model XLI.—For the narrow flat-top crests the disagreements become very large. In Model XLI, with a width of 1.65 ft., the maximum disagreement is between Messrs. Williams and Horton for a head of 1 ft., the difference being about 8%, and the disagreement between Messrs. Williams and Lyman for a head of 3.5 ft. is nearly as large.

If differences in the interpretations of the experiments running as high as 13% are justifiable, the writer wishes to ask what degree of accuracy may be expected in practice in calculating the discharge over a broad-crested weir, or in designing a weir to produce a given discharge? No doubt others who have occasion to use these results would be glad to have information on this point, and it is hoped that the author will be able to explain the discrepancies above noted so that the results of these experiments may be used with a greater degree of confidence than now seems warranted.

Allen Hazen, M. Am. Soc. C. E. (by letter).—The Society is indebted to Mr. Lyman for an extremely interesting and ingenious comparison of the most important weir measurements made by various experimenters up to the present time. Plate XXII, showing the percentage variation of all the different experiments, gives a more concise view of the whole subject than is to be obtained elsewhere. Plate XXI, showing the discharge over weirs, is most ingenious, and may be accepted as an authoritative compilation of the best existing data. The methods of plotting by which small differences are shown on a larger scale is to be taken as by no means the least of the good things in the paper. Mr. Hazen.

Notwithstanding the use of exponential formulas and the author's statements in regard to them, the writer believes that there are still advantages in the use of a formula which can be solved with an ordinary Mannheim slide-rule, where the conditions permit a reasonable accuracy to be so obtained. In the case of weirs, this can be done, because the discharge over weirs follows more or less closely the 1.5 power of the head, and the 1.5 power can be obtained rapidly and accurately with the slide-rule.

There is one fundamental proposition in regard to weirs that the writer has used for years and believes to be correct, which does not

Mr. Hazen. enter into the analysis of the data used by the author. This is: that two weirs of different size but of the same relative proportion in all their parts, that is to say, with the same ratio between the height of the weir, the depth of water over the weir, and the distance back from the weir that the head is measured, and with crests (if not standard sharp-edged crests) of the same proportionate dimensions, will always give discharges in proportion to the 1.5 power of the heads. In other words, such similar weirs of different sizes will have the same value of C in the formula, $Q = C\ h^{1.5}$. As far as the writer knows, the only evidence of substantial divergence of experimental results from this relation is in the case of a few experiments with very low heads. It may be that the viscosity of water in these cases is a disturbing element and accounts for these divergencies. We know, from data on the flow of water in pipes, that the viscosity of water has only to be reckoned with at low velocities, and disappears rapidly and completely as the velocities are increased. If these divergencies with low heads are really due to the viscosity, no appreciable effect from it is to be anticipated at the higher heads for which calculations are most frequently made. The fraction, $\frac{0.0148}{h}$, in the Bazin formula, and the arbitrary addition of 0.007, in the Fteley and Stearns formula, are the practical indications of these divergencies at low heads. Except for this feature, both the Bazin and the Fteley and Stearns formulas are based on the above stated proposition as applied to a standard weir.

If the proposition is accepted that weirs which, in all respects, are proportional to each other will have the same coefficients, then the distance back from the weir at which the head is measured should also be in proportion to the height of the weir. Using an arbitrary distance of 15 ft. seems to be illogical; and for very small weirs most inconvenient. Following Fteley and Stearns' statement, the minimum distance for such measurement is to be taken as 2.5 times the height of the weir. As both convenience and accuracy will be favored by as short a distance as is permissible, the writer suggests that it would be better to measure the head in all cases at a distance back from the weir equal to 2.5 times the height of the weir.

Francis, in deducing a formula from his experiments, used an arbitrary method of allowing for the velocity of approach, which he stated was probably below the truth. As the velocities to be allowed for in his experiments were always low, the influence of this error on the formula which he deduced was not great. Afterward, Fteley and Stearns proposed a method of allowing for the velocity of approach which was based on their own carefully made experiments on this particular point. They pointed out that, if the Francis results were recomputed with the allowance which their experiments indicated to be proper,

the results obtained were practically identical with their own. In other words, the experimental results of Francis and of Fteley and Stearns were completely in accord. Mr. Hazen.

The Bazin formula differs from these two in making a much greater allowance for the deviations at low heads, and in providing a more convenient means for taking into account the velocity of approach. The allowance in it for deviations at low heads is much greater than is indicated by Fteley and Stearns' experiments, the latter seeming to have been more exhaustive on this point than were those of Bazin. Between these two formulas, that of Fteley and Stearns would seem to deserve preference for general use, were it not for the cumbersome and inconvenient method of allowing for the velocity of approach.

The deviations at low heads can better be considered as a separate matter. Considering the relative ease with which experiments to determine the discharge at low heads can be carried out, it is surprising how little exact knowledge exists as to these quantities, and the influence on them of the temperature and the character and exact shape of the lip.

It may be that the desire to produce a formula which would account for the results at low heads as well as at high ones has led to greater complexity than is really necessary in a formula to be used only for higher heads. To see if it would not be possible to find one of more simple form, which would account for the data, a number of formulas were tried by the writer. The general form, $Q = \left(k + a\,\frac{h}{p}\right) h^{1.5}$, was selected as worthy of further study. From Fteley and Stearns' experiments, the approximate value of a was found to be 0.5; then $Q = \left(k + 0.5\,\frac{h}{p}\right) h^{1.5}$, in which the value of k remains to be determined. The values of k in the formula, in order to account for the various experiments of Francis, and of Fteley and Stearns, with weirs without end contractions and with heads of more than 0.3 ft., were found by approximate slide-rule calculations, and these results are shown in Fig. 48.

In the experimental results of Fteley and Stearns, used to show the effect of different heights of weir, the absolute quantities are not given. In each of these cases the first experiment, that is, the one with the highest weir, was used to compute the quantity of water for all the experiments in that series, and from this quantity the values of k for the remaining experiments in that series were computed.

On Fig. 48 have also been plotted some values of k corresponding to the exact solutions of the Fteley and Stearns and Bazin formulas, for conditions which are indicated. There is also drawn a straight line corresponding to the value of k which best represents all the

Mr. Hazen.

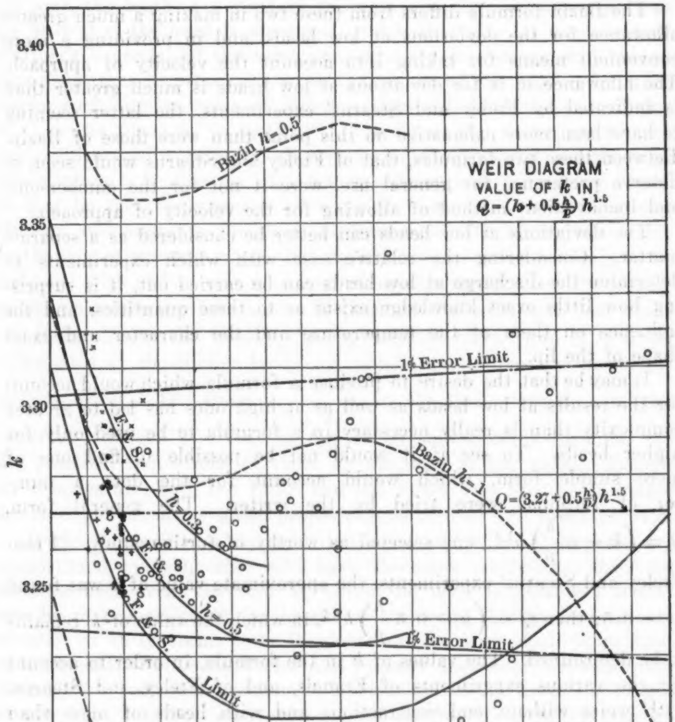


FIG. 48.

experimental data, and also lines showing 1% errors or variations in quantity of discharge from the line drawn. This method of plotting is somewhat similar to that used by Mr. Lyman, to whom acknowledgment is made. Mr. Hazen.

It is seen from all this that the formula, $Q = \left(3.27 + 0.5\,\frac{h}{p}\right) h^{1.5}$, accounts for all the Francis results, and nearly all the Fteley and Stearns results where the head exceeded 0.3 ft., within 1%, and it seems to serve quite as well in this particular as any formula that has been proposed.

The Bazin formula indicates a much wider divergence between the coefficients with high heads and low heads, and in this respect is not supported by the Fteley and Stearns experiments.

This simplified formula, which produces results that differ but slightly from those obtained by the Fteley and Stearns formula when velocity of approach is allowed for, has the great advantage over it that it can be easily solved on a slide-rule, automatically allows for the velocity of approach, and accounts for all the experimental results with heads greater than 0.3 ft. as well as the original formula. As compared with a diagram like Mr. Lyman's, it has the advantage of allowing results of all needful accuracy to be taken off for intermediate values of p and h for which interpolation in even so good a diagram as that of the author may sometimes lead to greater errors than are desirable.

Clarence T. Johnston, M. Am. Soc. C. E. (by letter).—It is fortunate that a man qualified to discuss the measurement of flowing water has undertaken a study of this subject. Mr. Lyman has brought together the available material relating to the actual measurement of water flowing over weirs of various kinds, and the diagrams and formulas he has presented must be very helpful to the Engineering Profession. Mr. Johnston.

His investigations have led him to recommend the rectangular weir without end contractions, and he advises legislation which will make this form of weir a standard. This would seem advisable, assuming that weirs can be used wherever flowing water is to be measured. The streams of Utah and the rivers and many of the ditches and canals of the West have sufficient fall to make weirs generally feasible. In many places, however, the fall is insufficient, and there, the measuring flume, which is rated by actual current-meter gaugings, must be used. Those who have used weirs extensively know how difficult it is to maintain them so that they furnish uniformly correct results at all times. It might be possible, in some cases, to place the weir in a channel running parallel to the canal or natural stream and have gates which would enable those in charge to divert the water over the weir only

Mr. Johnston. when measurements were being made. This would avoid some of the troubles that generally arise. Under these conditions, the structure in which the weir is placed can be inspected, and those in charge can ascertain whether or not water may pass around or under it. They can clean the channel prior to the beginning of any test, thus maintaining a constant height of the crest of the weir above the bed of the waterway. The natural channel up stream from the weir will not be changed greatly by erosion, if water runs through it only at intervals. It would seem that, in establishing a legal measuring device, precautions should be taken so that such device may have general application and so that, wherever it is used, engineers would have general confidence in the results obtained.

Plate XXI, and those following, relating to the discharge of various kinds of weirs when the head and the height of the crest are known, are the most satisfactory diagrams of the kind thus far published.

Plates XXVII*a* and XXVII*b*, together with the text describing them, deserve special mention. The text, even with the examples given, does not show the basic theory of the method used. It is possible that Mr. Lyman feels that the underlying theory is so elementary that a discussion thereof is not advisable. Under such an assumption, however, his numerical examples and some descriptive matter might have been eliminated.

He takes for his general equation, $Q = mh^n$, which, when reduced to logarithmic form is $\log. Q = n \log. h + \log. m$. The latter equation. when plotted on rectangular axes, represents a straight line, as he mentions. Certain measurements have been made in practice, which establish points referred to these axes in terms of log. Q and log. h. Approximate values of m and n are known, so that it is possible to substitute these values in the foregoing equation, and this Mr. Lyman calls the "trial" formula. When the actual measurements are considered, it is found that, for any value of log. h, the discrepancy between the measured and computed values of log. Q is very small. In order to show the discrepancies, the values plotted for log. Q must be shown on a very large scale. This is almost impossible. In addition, the lines lie so close to the axis of log. Q that the intersections with that axis are hard to locate. Mr. Lyman says: "Hence, the differences in the values of log. Q cannot be shown with sufficient accuracy on a drawing of reasonable size when rectangular axes are used." This is an accurate statement of the situation, except that there is no apparent reason for using axes which are not rectangular, and it would seem to one who has not given extended study to the problem that Mr. Lyman continues to use rectangular axes.

Mr. Johnston.

Referring to Fig. 49, the logarithmic equation has been plotted on the rectangular axes, log. h and log. Q. The equation takes the form

$$\log. Q = \frac{a+b}{c} \log. h + b \text{.................} (I)$$

The scale to which the values of log. h and log. Q are plotted, may be fixed to suit the problem under consideration. In Fig. 49, AB represents the line established by the "trial" formula. It is impracticable to plot values of log. Q to the fourth decimal place, measuring from the axis of log. h. If this were feasible, the values, a and b, would be shown on the same scale and for any values, k, k', etc., for log. h and S, S', etc., for log. Q, the location of the line showing the relation of log. h and log. Q, as measured, could be established. Instead, we subtract the values of log. Q as measured from those computed for various values of log. h. This gives us s, s', etc. These are

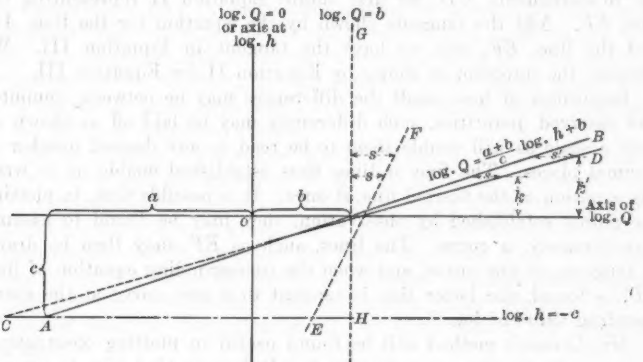


FIG. 49.

shown on Fig. 49, and they might be laid off from the line representing the "trial" formula as before, if it were not for the objections stated. Instead of laying off these differences from that line, we will lay them off from the line log. $Q = b$, as shown in the diagram. This locates the line, EF. This line is actually located by drawing a straight line in such a way as to approximate a number of the points established by the measurements. Two points, however, fix the line, and it is always possible to select two points on the line after it is drawn, regardless of how many measurements it may represent. The equation of the line, EF, is:

$$\log. Q = \frac{s' - s}{k' - k} \log. h + b + \frac{k' s - k s'}{k' - k} \text{..........} (II)$$

when referred to the same rectangular axes.

Mr. Johnston.

The equation desired represents the line, *CD*. Referring to the diagram, it will be seen that this line passes through the points, the co-ordinates of which are log. $Q = \frac{a+b}{c} k + b + s$, log. $h = k$, and log. $Q = \frac{a+b}{c} k' + b + s'$, log. $h = k'$. The equation of a straight line through these two points is

$$\log. Q = \left(\frac{a+b}{c} + \frac{s' - s}{k' - k}\right) \log. h + b + \frac{k' s - k s'}{k' - k} \quad \ldots . \text{(III)}$$

Now, in examining Equations I, II, and III, we find that the tangent of the angle between the line represented by Equation III and the axis of log. h is the sum of the tangents of the other two lines, and that in both Equations II and III the intercepts on the axis of log. Q are the same. Therefore, to find the equation of the line representing the measurements, *CD,* we first obtain Equation II representing the line, *EF*. Add the tangents shown by the equation for the line, *AB,* and the line, *EF,* and we have the tangent in Equation III. We preserve the intercept as shown by Equation II for Equation III.

Regardless of how small the differences may be between computed and observed quantities, such differences may be laid off as shown on such a scale as will enable them to be read to any desired number of decimal places. The line or lines thus established enable us to write the equation of the desired line at once. It is possible that, in plotting the points established by observation, they may be found to assume, approximately, a curve. The lines, such as *EF,* may then be drawn as tangents to this curve, and when the corresponding equation of line, *CD,* is found, the latter line is tangent to a new curve at the corresponding value of log. h.

Mr. Lyman's method will be found useful in plotting observations in many kinds of engineering work. It has enabled him to present weir formulas covering a wide range of experiment. His equations and diagrams doubtless will furnish the hydraulic engineer with better tools than have hitherto been provided.

Mr. Robinson.

R. B. Robinson, Assoc. M. Am. Soc. C. E. (by letter).—The writer heartily agrees with Mr. Lyman—and believes that every Western engineer will—that a legalized standard device and regulations to govern the measurement of water in flowing streams are very much needed, and that such device and regulations should be of the simplest possible form and construction consistent with correct and practical results. He would also go further, and say that the device and regulations should be designated by Federal statute, in order to avoid possible complications owing to different State localities on the same stream.

If this comment may serve to add in any way toward the accomplishment of Professor Lyman's aims, as set forth in his paper, it will have answered the purpose for which it is written.

L. M. WINSOR, ESQ.* (by letter).—One of the most crying needs to-day of the entire irrigated West is careful attention to accurate methods of measuring the flow of streams and to the proper distribution of this flow. Through the continual development of larger areas under irrigation, water has become or is rapidly becoming much more valuable than the land it covers. Water users are no longer satisfied with the loose methods of division which were put into practice during the early years of irrigation development. Each season brings new demands for an accurate system of measurement and distribution. For this reason it is high time for the Engineering Profession to perfect some accurate, economical, and at the same time simple, device to meet this demand. Mr. Winsor.

The weir in its various forms has been accepted as the device best suited to the needs of the irrigator. Before adopting it as a standard, however, it would seem advisable to make a more careful study of its use, under actual field conditions, than has ever been made. Most of the observations thus far have been conducted under conditions quite foreign to those which are met in the operation of a canal system. The early experimenters used, in the main, larger streams than the ordinary irrigation stream requiring measurements, and the recent experiments were conducted under laboratory conditions which are seldom applicable to the ordinary irrigation stream.

The work being done at the new laboratory at Fort Collins, Colo., under the immediate direction of Mr. C. M. Cone, of the Division of Irrigation Investigations, U. S. Department of Agriculture, will without question throw a great deal of new light on the matter of water measurement, and is no doubt the most important work of its kind going on to-day. This laboratory is equipped to make measurements under practical conditions, and the results there obtained will be of untold value to the irrigators of the West, and to the irrigation engineer.

The sharp-crested rectangular weir without end contractions, recommended in the paper, is without doubt one of the best forms, if not the best, now in use, where it can be put in place properly. It is not, however, as easy to erect for ordinary use as is the weir with end contractions. The latter can be placed directly across the channel without the trouble or expense of constructing a channel of approach, the only requisite being the enlargement of the up-stream portion of the stream bed to a sufficient area. In practice, the weir, or any other device, must be adopted by degrees. Farmers or canal operators object at first to any device which is expensive; therefore it is well to give due consideration to one which will give results approaching accuracy, and at the same time admit of construction. When canal operators are educated to the proper point, the most accurate of the various devices may be recommended, even if expensive.

*Asst. Irrig. Engr., Office of Experiment Stations, U. S. Dept. of Agriculture, Logan, Utah.

Mr. Winsor. In any case, if the suppressed weir is to be used, it is very important that the area of the up-stream channel be enlarged considerably in order to reduce the velocity above the approach to the weir to such an extent that it will not affect the flow.

The point made by the author that the velocity over a suppressed weir is greater than that over a contracted one is very important when applied to most western streams, which carry a great deal of sediment. This deposit of sediment above the weir is one of the main factors in prohibiting its use under irrigation conditions.

In the writer's opinion, the discussion of the various forms of broad-crested weirs, as applied to the present needs of the country, is less important than the portion devoted to the sharp-crested weir, and particularly to the first three plates or diagrams. Great credit is due the author for his originality in working out this system, and particularly for his clear and concise comparisons between the various experiments.

Mr. King. H. W. King, M. Am. Soc. C. E. (by letter).—The weir experiments of Francis, Fteley and Stearns, and Bazin have been examined so minutely, and have been grouped and studied so carefully and thoroughly by so many able hydraulicians that it would hardly seem possible to evolve anything really new from them. The author, however, by his method of combining into working diagrams the results of these experiments and the more recent ones of the Cornell and University of Utah Hydraulic Laboratories, has done much to simplify the use of this mass of data. The contribution is a most valuable one, because of the new methods of analysis developed, as well as the practical value of the results obtained.

Inasmuch as the author did not endeavor to derive a general formula of discharge, he was able to obtain several formulas for each series of observations which fit very closely each value given. These exact formulas were in turn used in deriving the quantities used in the diagrams. It is evident that the author is consistent in his methods of constructing these diagrams, and they represent accurately the results obtained from the experiments. There can be no doubt that quantities can be picked off the diagrams with as great accuracy as is justified by the results of the original experiments, and, doubtless, they furnish the most ready and reliable means yet provided for solving problems of the type to which they apply.

This paper, however, should be considered as a very comprehensive study of one phase of the subject of weirs rather than a general analysis. The author takes up the single case of a weir in a channel of rectangular cross-section in which the length of the weir is equal to the width of the channel. This is the condition under which many, if not most, of the weir experiments above referred to were made. In constructing a weir for the measurement of water, it fre-

quently happens that these requirements may be fulfilled. It is not true, however, that this may be done in all cases. In many cases the engineer wishes to measure the flow of water by the weir method in a canal of trapezoidal section or a stream of irregular section. There is, moreover, a question as to whether it will be practicable under all conditions to require the measurement of irrigation waters by suppressed weirs in rectangular channels, as the author suggests. These ideal conditions should be obtained when practicable, but there seems to be no good reason for limiting the use of the weir to this one case. Mr. King.

Occasions will continually arise when it will be desirable to construct weirs across other than rectangular channels, and there will consequently continue to be need for an accurate solution of these problems. It also appears that we are not yet ready to dispense with the weir with end contractions, and, therefore, there is still need for a means of determining the discharge over such weirs. It is doubtless true that, for maximum accuracy, with our present limited data and knowledge regarding the flow over weirs, the channel of rectangular section and suppressed weir should be used where practicable. This, however, is due to the fact that the experimental data thus far obtained apply more particularly to weirs of this type, and not because those constructed under other conditions may not be possible of as exact solution.

In order that a method of solution of weir problems may be of general application, it must provide for a channel of approach of any cross-section. It should also, with proper modifications, include the weir with end contractions. It would appear, therefore, that the foundation of the method of solution must apply to the weir discharging from the theoretically quiet pond. The coefficient of discharge under such conditions is well known to be very close to a constant quantity within the range of head covered by our present experimental knowledge. The deviation of this coefficient from a constant, therefore, is due almost entirely to effects of velocity of approach. The present weir formulas, with the exception of Bazin's, use this general method of solution; that is, they determine the discharge from a still pond and correct the observed head, so as to include the increased discharge due to velocity of approach. That these formulas do not give results of sufficient accuracy is unquestioned. The trouble, however, does not lie in erroneous theory or assumptions, but in being based on insufficient if not partly unreliable experimental data.

There still remains to the Engineering Profession the problem of providing a satisfactory general method of solution for all weir problems. Such a method of solution must be based largely on new data, which should cover a wider range than those existing, and be of

Mr. King. unquestioned accuracy. New experiments are needed to give more definite knowledge of the discharge over weirs without velocity of approach, which will replace the assortment of formulas in use at present. There is even greater need of complete data which will give a clear understanding of the laws governing the flow of water, the distribution of velocities in the channel of approach, and the effect of these laws on the discharge over weirs. At present, adequate facilities for such experiments are not available, but it is to be hoped that a means will soon be provided for obtaining these much needed data. With them a ready means should be provided for solving all weir problems within the required limits of accuracy.

Mr. Horton. ROBERT E. HORTON, M. AM. SOC. C. E. (by letter).—The writer regrets that he has been unable to find time to give this paper the careful perusal it undoubtedly deserves.

Heretofore, a thin-edged weir, constructed substantially in accordance with the rules laid down by the late James B. Francis, Past-President, Am. Soc. C. E., has commonly been designated a "standard weir", and has often been accepted by the Courts as a correct method of measuring water, whether or not it was so in fact. The writer agrees with Mr. Lyman to the extent that a suppressed weir is an equally good "standard", but questions the propriety of legalizing this or any other form of weir as a "standard". It seems to the writer that there are now a considerable number of forms of weirs, all sufficiently well calibrated to serve as excellent standards in particular cases, no one of which is the best standard for all cases.

The hydraulics of weir discharge is by no means as simple a matter as it may appear to a neophyte. A good engineer can get good results with good weirs of various forms. The writer does not hesitate to say that he has seen "Francis Standard" weirs abused—and the results sworn to in Court—in a startling manner. Such abuses are usually coupled with the opinion, tacit at least, that as the weir is denominated "standard", it is consequently fool-proof, and the element of personal equation is eliminated.

Great injury has been done by writers of textbooks on hydraulics by placing undue stress on the merits of so-called standard weirs and orifices, especially the latter, as compared with other forms. Almost any kind of a round-edged orifice, with a reasonable edge radius, is subject to less variation in coefficient in practice, and is less likely to be obstructed or change its form, than a thin-edged orifice. The same is true to some extent for weirs. It is also true in both cases that the round-edged form discharges more water for an opening of a given size then the thin-edged form. This is a matter of some importance, in the case of orifices, in measuring the supply of water-wheels through flume or bulkhead openings where water is sold or leased in square inches or other units. It is a matter of great importance in design-

Mr. Horton.

ing flood spillways for dams, because, with a given length, the round-crested spillway, having a larger coefficient, will discharge a flood with a less depth of back-water than a broad flat or a thin-edged weir. The floods in the spring of 1913 demonstrated pretty clearly that spillways of dams form the best practicable method of measuring the maximum flood discharge of rivers, and for that purpose, ogee or round-crested weir sections are the best practicable standard.

Mr. Lyman describes various base formulas which, with suitable coefficients, have been used for calculating the discharge over weirs of various forms. In the original deduction of the U. S. Geological Survey experiments of 1903*, the writer chose the theoretical formula, which had also been adopted as a base formula by Francis for thin-edged weirs:

$$Q = C\,L\,H^{\frac{3}{2}}$$

The reasons were as follows:

(1) The results in this form for irregular sections are directly comparable with the discharge of a thin-edged weir, when the latter is calculated by the formula then in most common use.

(2) The coefficients for many forms of weirs of irregular section are not continuous functions of the head, so that the full history of the discharge under a wide range of heads cannot be represented by any single formula with a constant coefficient.†

(3) If, as is true in many, but by no means all, cases, a variable coefficient must be used in formulas for such weirs, then it seems better to throw all the variation into the coefficient and make the computation as simple as possible by using the $\frac{3}{2}$ power of the head, instead of causing the head to vary according to some other fractional or decimal power, which would require the use of logarithms, logarithmic paper, or a logarithmic slide-rule for practical computations.

Judgment based on experience is one of the best checks on computations, and is often a valuable criterion as to the reliability of experimental results, and, for this reason, the writer prefers to compute experimental results in the first instance in some manner that will make such results commensurate with some established scale or "yard-stick", as the Francis formula for weirs, or the Chezy-Kutter formula for channels.

If the results appear reasonable, they may afterward be applied to any formula desired. Francis applied an exponential formula to the Merrimac River Dam.

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* As described in Water Supply Papers Nos. 150 and 200. The table of multipliers appearing at the end of these papers was inserted after the paper left the writer's hands, and without his sanction.

† The presence of *points d'arret*, cusps, etc., in weir coefficient curves is shown very clearly in some of Bazin's experiments. See the writer's discussion of the paper by the late George W. Rafter, M. Am. Soc. C. E., entitled "On the Flow of Water Over Dams", *Transactions*, Am. Soc. C. E., Vol. XLIV, p. 345.

Mr. Horton. The writer early recognized the value of the exponential formula for certain forms of weirs, and applied it where it seemed to fit. Later, Mr. Philip Parker, using the writer's computation of the Bazin, U. S. Deep Waterways, and U. S. Geological Survey experiments, deduced exponential formulas for a large number of weirs.*

The writer is not in sympathy with the practice adopted by some, of working out a table of multipliers to be applied to the calculated discharge over a thin-edged weir to obtain the discharge over an irregular crest-section. First, the discharge over the weir in question might as well be calculated in the first instance, thus saving a deal of computation, with chances for error; second, for a given head and cross-section, the velocity of approach correction varies with the discharge coefficient, so that for two weirs, one thin-edged and the other of irregular section, with the same measured head, but with different sections of approach, the use of multipliers becomes confusing and may be a source of error.

The use of either the Francis base formula or the exponential formula, both with varying coefficients where necessary, seems to the writer better practice, for cases where there is no abrupt change in form of nappe, as when the water breaks free from a flat crest and assumes the thin-edge weir form, the exponential form of expression probably gives in many cases the simplest and most complete history of the discharge.

In conclusion the writer is prompted to remark, regretfully, as regards hydraulic science, "Of the making of many formulas there is no end."

Mr. Smith. G. E. P. SMITH, M. AM. SOC. C. E. (by letter).—The author's discussion of weirs is particularly timely, inasmuch as the necessity for the measurement of water by all water users is being felt at the present time. There are comparatively few irrigation projects in the Southwest where the water is not now delivered, nominally at least, by measure.

The author has rendered valuable service in collating the various masses of experimental data on the flow over suppressed weirs. There appears to be a general agreement among experimenters within 3%, though a divergence of results as high as 5% is not infrequent. Apparently, there is need of another set of experiments in which conditions and methods are defined much more rigorously than any thus far attempted. If an important new set is made in the future, doubtless the author will wish to amend his formulas. On account of the nature of the problem, no formulas can be accepted at the present time as final.

* A few of Mr. Parker's formulas appear in his recent work "The Control of Water."

The Francis formula, a sort of average law, has been disintegrated into a multitude of formulas, each covering a limited range of conditions. This is in the interests of accuracy, and here the author has succeeded very well, but the writer cannot agree with him that it is in the interests of simplicity. Of course, the general use of accurately platted diagrams avoids the use of cumbersome tables, but errors in taking results from the diagrams can easily exceed the discrepancy between the author's and the Francis formulas. Mr. Smith.

One assumption made at the outset of the paper is perhaps open to question. It is that the discharge is proportional to the length of crest. The influence of the side-walls on the discharge must be variable, depending on the material of which the walls are built and the character of the workmanship. The influence is comparatively slight for long crests, but of considerable importance in the case of short ones, such as are found in irrigation laterals and head-ditches. A serious difficulty in applying the author's method is the standardization of the side-walls. It is much simpler to standardize an edge than a wall.

It would seem to be unnecessary, too, to place the gauge so far up stream as is recommended. The object being to get back of the "surface curve," which depends mainly on the head, it is more reasonable to place the gauge at a distance back dependent on the head under ordinary or average conditions of the stream, say, at a distance equal to four times the head.

The writer formerly used the standard rectangular weir, but abandoned it gradually in favor of the Cippoletti weir, on account of the greater ease and simplicity of installing the latter. The Cippoletti weir requires much less construction than a suppressed weir, a pool being formed up stream in place of a flume. On one occasion the writer built and set an 18-in. weir, got a satisfactory reading and caught a train in 80 min. The Cippoletti weir discharge is proportional to the length, and a table of discharges is so simple that an engineer is not required to interpret it. Probably the greatest error in its use is in neglecting the velocity of approach, and this error can be minimized by providing an ample height of weir, and by applying the usual Francis correction. Furthermore, it would be possible to analyze the Cippoletti weir in a manner similar to the author's, and the subsequent use of the "approved" tables would result in the highest degree of accuracy possible with weirs.

Whatever doubt the writer had regarding the accuracy of the Cippoletti weir was set at rest by observing an exceedingly valuable set of experiments in the hydraulic laboratory of the University of Wisconsin. These tests were performed by senior students under the direction of Mr. G. J. Davis. They indicated that:

Mr. Smith.

1. The ratio, 1:4, is correct for the side slopes;
2. The coefficient, 3.37, is correct for crests more than 6 in. long;
3. The error of the formula will be less than 1%, if the head does not exceed 0.4 of the length of crest.

The restriction sometimes placed on the rectangular weir is that the head shall not exceed one-third the length of crest.

There is one idea expressed in the paper to which exception should be taken. The proposition to establish one type of weir for universal use by legislation is debatable at least. Legislators are not engineers, and the fewer legal restraints thrown around the Profession the better it will be for the Profession. There are no laws fixing the power of the transit telescope, or the size of sewer pipe, or steel rail sections, nor are such laws wanted. The standard unit of measure should be fixed, as it is, but the method, never. Better methods will be found in the future, and should not be debarred from use in advance. There are several good ways to measure land, to measure electric current, and to measure water. The best method for a particular case depends on the surrounding conditions.

Utah has a law establishing standard cross-sections for the roads of the State. Is it possible that the southern counties and the northern counties require the same road crowning and the same shape of gutters? In Arizona roads are built variously of volcanic cinders, gravel, out-wash deposits, caliche, and sand; some roads are used almost wholly by automobiles, others by burros; some are in pine forests, others where the rainfall averages less than 3 in. per year; some withstand traffic principally, others rain and frost, others terrific winds. Surely no standard cross-sections are applicable here. Logan Waller Page, M. Am. Soc. C. E., states that "individual judgment is necessary" to determine the cross-sections of roads.

Instead of more standardization, perhaps more originality is needed. Engineers sometimes follow the well-beaten track when much could be gained by taking the cut-off—a statement which is worthy of the following illustration. For several years the writer had charge of a weir in a mountain canyon near Tucson. The discharge of the canyon varies from a small trickle in the dry months of the drier years to occasional floods of more than 1 000 sec-ft. It was quite as essential, perhaps more so, to determine the discharge of the years of minimum run-off as that of the infrequent maximum years. What was needed then, was an adjustable length of crest. Another consideration was the shape of the canyon; there were vertical rocks on the north side, at the foot of which ran the small flows, and a rising bouldery stream-bed from there to the south side. The weir crest, 65 ft. long, was designed to fit the canyon. For 5 ft. on the north side it was level,

and the remaining 60 ft. had a slope upward of 1 in 15. Consequently, the small flows were concentrated over the short-level portion. The weir was built for one-third of what it would have cost with a crest of standard shape. The discharge formula for the weir is derived by using Francis' formula as a basis, and integrating separately for the triangular portion. The differential equation for the latter is Mr. Smith.

$$d\ Q = 3.33 \left(\frac{x}{15}\right)^{\frac{3}{2}} dx$$

and the discharge for the entire cross-section is given by the equation,

$$Q = 16\tfrac{2}{3}\ H^{\frac{3}{2}} + 20\ H^{\frac{5}{2}}.$$

The equation was tabluated and also platted as a curve. Generally, the curve was used in working up the nilometer records.

There were errors, of course, due to neglect of the velocity of approach, and, probably, to submergence during extremely large floods. The influence of the former could have been approximated, but it was desired to have any possible error on the safe side. The action of the flow was such as to keep the up-stream side scoured out to a depth of from 1 to 2 ft. Records were kept for 9 years, during which time the nilometer box was submerged three times. The credit for the design of the weir is due to Mr. S. M. Woodward, of the University if Iowa.

The writer, while traveling in the irrigated portion of Italy, saw many different methods of measuring water, among them various forms of weirs, submerged weirs, submerged orifices, and partly submerged orifices. It was interesting to observe that, in most cases, the method used was best fitted to the conditions of fall, discharge, accuracy desirable, and relative cost.

In the western part of the United States it is likely that a modification of the Venturi water meter will come into use to a considerable extent.

Clarence S. Jarvis, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Lyman's paper, and the discussions regarding stream measurement, are of unusual interest, and will be of much practical value, especially in the irrigated districts. Mr. Jarvis.

Mr. Lyman has admirably disposed of the stupendous task of correlating a mass of data collected under a wide range of conditions, with varying viscosity of the water due to temperature changes and to chemical elements held in solution or suspension, the slight variations in the intensity of the force of gravity, and the personal equations of the observers, as disturbing factors.

The graphical comparison, portrayed so clearly on Plate XXII, of actual discharges measured by eminent experimenters with the quantities derived by the use of the diagrams for sharp-crested rec-

Mr. Jarvis. tangular weirs without end contractions, proves to be a very satisfactory test of the curves on Plate XXI.

If a single type of weir were to be adopted, either the Cippoletti or the sharp-crested rectangular weir without end contractions would doubtless serve best in irrigation practice.

One advantage of the Cippoletti weir is illustrated by Robert S. Stockton, M. Am. Soc. C. E., in discussing the "Management of Irrigation Systems".* A portable sheet-steel Cippoletti weir, of 24-in. crest, is used by the ditch rider to measure the water supply in the various small laterals.

In many irrigation districts where the slope of the country is slight, the writer has observed the need for a combination of measuring flume for current-meter gaugings and the approach to a suppressed weir. During the period of high water the canal or lateral is taxed to its capacity, and the obstruction required to give a free fall for such a stream over a weir is prohibitive. During this period current-meter gaugings are in order; and the rating curves for successive years need not change materially.

With the beginning of the low-water period, which, in the Great Basin region, is generally early in July, the duty of water is often increased two- or three-fold, and, with the reduced stream in the canal, it is possible to insert the weir crest in the measuring flume, thus providing a free overfall. The sediment does not interfere seriously after the high water has ceased; and the increased accuracy and reduced labor involved in the weir measurements recommend this method. Under these conditions the rectangular weir without end contractions provides the maximum width of crest and the minimum obstruction to the flow in the canal, and is much to be preferred.

In the writer's opinion, the data contained in Mr. Lyman's paper, and in the discussions, fill a long felt demand, and should wield a strong influence in the direction of higher duty for irrigation water, which will result from its careful distribution and judicious application on the irrigable land.

Mr. Williams. GARDNER S. WILLIAMS, M. AM. SOC. C. E. (by letter).—This paper brings to the front once more the measurement of water over weirs, and embodies the results arrived at by the author in a study of most of the data available in 1905, supplemented by some investigations of his own of more recent date. It is to be regretted that the hitherto unpublished data of experiments involved in this paper were not presented in such a concise form as to have warranted publication. In physical investigation, the one thing above all others that is valuable is a correct record of the experiments and their results. Opinions, theories, and conclusions are likely to change as time goes on, but

* *Engineering and Contracting*, January 28th, 1914.

physical facts remain physical facts, and may be as valuable 100 years hence as they are now, though the experimenter's conclusions may be long discarded. Mr. Williams.

A considerable part of the experimental work discussed in the paper was performed in the Hydraulic Laboratory of Cornell University under the writer's direction, and much of it under his personal supervision and with his participation, and the data have received careful study at his hands independently of the author's work.

Our basic knowledge of the quantity of water flowing over weirs rests on the experiments of Francis, Fteley and Stearns, and Bazin, all of whom experimented on weirs of different dimensions, and read the head of water at different places or by different devices. No volumetric measurements were made at Cornell prior to 1905, except in a few low-head experiments wherein the inaccuracy of the volumetric measurement, on account of the large area and small rise in the measuring canal, was greater than that of any of the weir formulas could possibly have been. Since that date volumetric measurements have been made at Cornell over a weir 6 ft. long, and at the University of Utah, as described by the author. It is to be hoped that the data of the Cornell experiments may be presented in the discussion of this paper, as the volumetric determinations were probably more accurately made than in any other series of investigations by reason of the measuring basin being a vertical tube or stand-pipe 6 ft. in diameter and about 60 ft. high.

The builders of the Hydraulic Laboratory at Cornell made no provision for the reading of heads on the weirs contemplated. To cut chambers in the solid concrete of the walls and the rock adjoining it involved considerable expense, and when the first weir investigation was called for, that of the U. S. Board of Engineers on Deep Waterways, after some discussion, a device for communicating the head to glass tube gauges was adopted, consisting of a 1-in. galvanized-iron pipe, perforated with ¼-in. holes every 6 in. throughout its length, and placed transversely across the channel of approach, with the openings on the under side, and the pipe 8 in. above the bottom. Three such pipes were set for each weir, varying in distances from the base of the respective weirs. For this device there was then the unquestioned precedent of the similar apparatus at the Holyoke testing flume, which had been used there for many years, and is still. Before this series of experiments was completed, however, the possibility of an erroneous indication of head suggested itself, and, to test the matter, similar pipes were set in the floor of the timber flume leading to the lower weir, with the openings on the top and just flush with the bottom of the channel. The result was somewhat startling, for it was found that, for heads of about 4.50 ft., the original device was giving an indication about 0.30 ft. lower than that shown by the openings in the bottom of

Mr. Williams. the channel. A series of simultaneous observations on the two devices was made, from which the former were corrected to the latter, and it was assumed that this latter device corresponded in its indication to those used by Francis, Fteley and Stearns, and Bazin. At the upper weir it was not so easy to get a series of openings flush with the bottom, as that would have necessitated cutting into the concrete, and after some study of existing data, the so-called standard tube-piezometer, described by the author on page 1263, was adopted, and on the strength of the experience of Desmond FitzGerald, Past-President, Am. Soc. C. E., with a perforated pipe in a water main,* it was assumed that the indication of this device corresponded to that of an orifice at the bottom of the side-wall similar to the one used by Bazin. With the correctness of this assumption the writer was never entirely satisfied, but, during his connection with the Laboratory, from the conclusion of the first series of investigations, this device was used at the upper weir in all experiments and very carefully observed, all other devices being compared with it. During the earlier work the upper weir was taken as the standard, and the discharge over it was computed by Bazin's formula, using the indication of the standard tube for the head. In the spring of 1901 the lower standard weir was installed, and the head on it was usually read by the 13-ft., three-tube-piezometer, described by the author on page 1271. The indication of this apparatus was never considered quite as reliable as that of the standard tube because of a greater likelihood of the presence of air in the pipe connecting it to its gauge. The lower sharp-edged weir was designed to be as nearly as possible a duplicate of the 19-ft., Farm Pond weir of Fteley and Stearns, it being 6.65 ft. high, or $\frac{1}{10}$ ft. higher than the Farm Pond weir, and 16 ft. long. During an extended series of experiments the head on this weir was read at a point 6 ft. up stream from the crest and just below its level, this coinciding with the practice of both Francis and Fteley and Stearns.

To those interested in the more precise measurement of water, the data obtained from the resulting use of different devices for reading head make the most valuable part of this paper, and are embodied in Tables A and B.

The first investigations, as to the effect on the apparent discharges, by different methods of reading the head on weirs, were made by the late James B. Francis, Past-President, Am. Soc. C. E., and are presented in "The Lowell Hydraulic Experiments".† These experiments are discussed by Messrs. Fteley and Stearns.‡

The next investigation of the subject of reading heads, so far as the writer has discovered, was that of Hiram F. Mills, Hon. M. Am. Soc.

* *Transactions*, Am. Soc. C. E., Vol. XXXV.

† Edition of 1883, page 187 *et seq.*

‡ *Transactions*, Am. Soc. C. E., Vol. XII.

C. E.,* which appeared to demonstrate that an opening in the wall of a channel, in order to transmit a pressure corresponding to the height of the surface of flowing water opposite, must be flush with the side, and the connecting pipe must be at right angles thereto. Mr. Williams.

The subject of indications of head, particularly as applied to closed channels, received further discussion, but without experimental evidence, in the paper on "Experiments Relating to Hydraulics of Fire Streams" by John R. Freeman, M. Am. Soc. C. E.†

In the paper on "Flow of Water in 48-In. Pipes" by Desmond FitzGerald, Past-President, Am. Soc. C. E.,‡ a statement appears covering the similarity of indication of a perforated pipe laid longitudinally on the bottom of a water main, and a series of circumferential orifices communicating to a chamber surrounding the pipe; and some further investigations along similar lines are presented in the paper on "Experiments on the Flow of Water in the Six-Foot Steel and Wood Pipe Line of the Pioneer Electric Power Company, at Ogden, Utah".§ Beyond this, the effect of different methods of reading head does not appear to have attracted special notice until the writer called attention to it and presented experimental data bearing thereon, in the discussion of the paper, "On the Flow of Water over Dams," by the late George W. Rafter, M. Am. Soc. C. E.,§ since which time nothing further has been presented in the publications of this Society, at least as applied to weirs, until the paper under discussion.

In these days, particularly in water-power work, where the competition between various manufacturers leads to the proposal of bonuses and forfeits, based on as small variations of discharge as one-tenth of 1%, exact knowledge of the influence of every departure from the standard of reference is of inestimable importance. In a recent test of water-power apparatus, a bonus of several thousands of dollars was apparently won, a considerable part of which was due to the introduction of irregular methods of observing the head on a weir, without a consideration of their influences.

Theoretically, what is desired in reading the head on a weir is to locate the elevation of the surface of the water, but, with channels of even very moderate widths, this will not be the same at all points across the stream, so that the measurement of it by even several tapes in a cross-section may not give an exact indication of the average head, and if it would, in no experimental investigation wherein the discharge of large weirs has been determined volumetrically, except those at the University of Utah, has such a process been used. The methods of transmitting the head through orifices in the side-walls or

* *Proceedings*, American Academy of Arts and Sciences, Vol. VI, New Series, Boston, 1879.

† *Transactions*, Am. Soc. C. E., Vol. XXI.

‡ *Transactions*, Am. Soc. C. E., Vol. XXXV.

§ *Transactions*, Am. Soc. C. E., Vol. XLIV.

Mr. Williams.

in plane surfaces parallel to them, as adopted by the leading investigators, is open to the criticism that the pressure thus transmitted is influenced by the velocity past the orifice to an extent that becomes decidedly appreciable at high heads. If, in order to avoid high heads, long weirs are used, the question then arises as to the applicability of an indication transmitted through the side-wall, to the establishment of the average head across a stream, say, 100 ft. wide. Bazin apparently conceived that the head should be transmitted from a location where the velocity would be the lowest possible, and hence selected a point well up stream from the pressure angle and at the bottom of the side-wall. For long weirs it seems to be necessary to supplement this orifice by openings in the bottom of the channel at frequent intervals across it, and perhaps the most satisfactory device is a chamber extending across the bottom of the channel and communicating with it through medium-sized orifices at frequent intervals. Although it may be assumed that the indication of head transmitted through such a device cannot be very different, for short weirs, from that used by Bazin, it is nevertheless quite certain that it will not exactly coincide.

Different forms of transverse and longitudinal pipes, variously perforated, are frequently used, but both devices, if outside the pressure angle, are sure to read lower than an orifice in the wall near Bazin's location, on account of the reduction of pressure due to increase of velocity, as the location departs from the wall and bottom. In the experiments presented by the author, the standard tube, although intended, and originally supposed, to measure a head corresponding to that indicated by Bazin's device, fails to do so, and actually gives a lower one. The 11-ft., wall-piezometer coincides more nearly with a Bazin indication, but is still, in all probability, too low, as by the author's equation the Bazin reading is more than 5% higher than the 15-ft. tape.

Being taken in the open channel, the tape readings are far from satisfactory for fine work. At low heads, with smooth water, the tendency of the observer is to cut the surface too deeply, thus giving a low head, and, for high heads and rough water, the observer invariably reads too high. The same applies to observations with the point-gauge in running water. This accounts in part for the irregularity of the lines representing tape readings. Although, if the water is stilled, a skilled observer can do excellent work with a tape or any other form of point-gauge, the ordinary student, dealing with the rough water encountered at high heads, comes very far from getting ideally consistent results.

In order that the record may be complete to date, the writer presents the following data of comparisons of different methods of reading heads.

I.—At Cornell University Hydraulic Laboratory, 1899. Upper Standard Weir, 16 ft. long. **Mr. Williams.**

A = Standard tube, by double-tube gauge, as described by the author.

B = Upper transverse pipe, by double-tube gauge.

This was a 1-in. pipe perforated on the bottom with ¼-in. holes, 6 in. apart, extending across the canal parallel to the weir, 8 in. above the bottom and 27 ft. up stream from the 13.13-ft., high, standard weir.

C = Middle transverse pipe, by double-tube gauge.

This was in all respects similar to B, except that its location was 10 ft. up stream from the weir.

A and B, and B and C were observed simultaneously on the same scale. Heads are from mean curves of the observations.

HEADS.

A	B	C
0.164 ft.	0.164 ft.	0.164 ft.
0.492 "	0.484 "	0.492 "
0.984 "	0.970 "	0.985 "
1.476 "	1.461 "	1.495 "
1.968 "	1.938 "	1.993 "
2.460 "	2.410 "	2.489 "
2.950 "	2.860 "	2.965 "

In these experiments it appears that C was probably within the pressure angle.

II.—At Cornell University Hydraulic Laboratory, 1899. Lower, Sharp-Edged Weir, 6.56 ft. long.

D = Flush pipe, by double-tube gauge.

This was a 1-in. pipe perforated on top with ¼-in. holes, 6 in. apart, and set with its top flush with the bottom of the flume, parallel to and 37 ft. up stream from the 5.2-ft., high, sharp-edged weir.

E = Upper transverse pipe, by double-tube gauge.

This was similar to B and 37 ft. up stream from the weir.

F = Middle transverse pipe, by double-tube gauge.

This was similar to B and 19 ft. up stream from the weir.

G = Middle flush pipe, by double-tube gauge.

This was similar to D and 19 ft. up stream from the weir.

D and E, E and F, and D and G, were observed simultaneously on the same scale. The heads for D, E, and F are from mean curves, those for D and G are from direct observations.

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HEADS.

D	*E*	*F*	*D*	*G*
0.920 ft.	0.912 ft.	*F* = *E*	0.744 ft.	0.745 ft.
1.010 "	1.002 "	Nearly.	0.913 "	0.914 "
1.061 "	1.050 "		1.086 "	1.088 "
1.632 "	1.614 "		1.292 "	1.292 "
1.668 "	1.642 "		1.441 "	1.442 "
1.970 "	1.934 "		1.632 "	1.631 "
2.566 "	2.480 "		1.810 "	1.810 "
3.380 "	3.270 "		1.811 "	1.807 "
3.820 "	3.655 "		2.240 "	2.239 "
4.160 "	3.970 "		2.642 "	2.643 "
4.310 "	4.080 "		(3.470) "	(3.460) "
4.820 "	4.560 "		4.270 "	4.277 "
5.040 "	4.720 "		4.670 "	4.681 "
5.450 "	5.090 "			
5.560 "	5.180 "			

III.—At Cornell University Hydraulic Laboratory, 1899. Lower Experimental Weir, 16 ft. long. U. S. Deep Waterways Section.* 1:1 up-stream slope tangent to 3.3-ft. radius curve at top; height, 6 ft.

H = Longitudinal pipe by double-tube gauge.

This was a 1-in. pipe similar to *B*, but laid on the bottom parallel to its axis and about 20 ft. up stream from the weir.

I = Series of ¼-in. orifices, 6 in. apart, throughout the length of the crest, by double-tube gauge.

J = Tape, nailed on the wall vertically from the crest of the weir, and read by contact of water surface.

H, therefore, indicates the head on the weir;
I, the pressure in the sheet at the crest; and
J, the vertical depth of water at the crest.

HEADS.

H	*I*	*J*
0.848 ft.	0.525 ft.	0.582 ft.
1.481 "	0.848 "	1.035 "
2.170 "	1.099 "	1.540 "
2.753 "	1.208 "	2.010 "
3.180 "	1.242 "	2.304 "

IV.—At the Holyoke Testing Flume, 1900. Sharp-Edged Weir, 20 ft. long.

* *Transactions*, Am. Soc. C. E., Vol. XLIV, p. 284.

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K = Francis orifice, by double-tube gauge.

This was a ¼-in. opening flush with side-wall of canal, 6 ft. up stream from, and 0.34 ft. below, the level of the crest of a weir 5.85 ft. high.

L = Holyoke pipe, by double-tube gauge.

This was a 1-in. brass pipe perforated on its under side with ⅛-in. holes every 2 in., and set 1 ft. above the bottom of the channel, parallel to and 10.2 ft. up stream from the 5.85-ft. high weir.

K and L were observed simultaneously on the same scale.

Heads.	
K	L
0.392 ft.	0.392 ft.
1.092 "	1.087 "
1.358 "	1.352 "
1.548 "	1.536 "
1.721 "	1.710 "
1.873 "	1.857 "
1.994 "	1.971 "
2.079 "	2.055 "

V.—At the University of Michigan, 1912. Special weir, 1.865 ft. long, with one end contraction of 1.08 ft. on side next orifices and gauge-reading devices. Weir, 11.46 ft. high.

M = Francis orifice, by hook-gauge.

This was ¼ in. in diameter, flush with side-wall, 6 ft. up stream from, and 0.34 ft. below, the level of the crest, connected by ¾-in. pipe to a gauge chamber 15 in. square.

N = Bazin orifice, by hook-gauge.

This was 4 in. in diameter, 7.32 ft. up stream from the weir, with its center 0.4 ft. above the bottom of the channel, connected by 4-in. pipe to a gauge chamber 15 in. square.

P = Point-gauge, its contact with the water surface observed electrically by the flash of a lamp.

This was situated in a 10 by 12-in. rectangular box of 2-in. plank attached to the side-wall above the Bazin orifice; the bottom of the box was fully open and 0.79 ft. above the level of the crest, and its down-stream face was 7.47 ft. up stream from the weir. The point was located 7.82 ft. from the weir, and 0.25 ft. from the wall side of the box.

The reading of the several gauges, corresponding to the crest of the weir, was determined both by wye-leveling and by water levels from a hook-gauge attached to the weir crest.

For M and N the crest reading by the water level was 0.005 ft. higher than by the wye-level, and for P, 0.011 ft. higher. This in-

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The heads observed were:

M	*N*	*P*
1.1551 ft.	1.1623 ft.	1.1758 ft.
1.3575 "	1.3883 "	1.3918 "
....	1.4572 "	1.4602 "
1.4386 "	1.4435 "	1.4456 "

It is to be noted that, on account of the small size of the opening for *M*, the water rose very slowly in the gauge tank, and the low reading of the second, as well as the relatively high reading of the fourth, observation may be partly due to that cause. The comparisons of *N* and *P* are believed to be very accurate.

As to the comparisons in the author's Tables A and B, it may be said that at the lower weir those of the Francis, float, and bottom piezometers, when more than one was used, being read by the same observer on the same scale of a double-tube gauge, should have been, and in all probability were, very carefully observed, and errors in one would be duplicated in the other.

The same is true as to comparisons at the upper weir between the 6-ft. longitudinal and 6-ft. wall, the 6-ft. longitudinal, the 11-ft. wall, the 2-ft. wall, and the crest piezometers which were observed in pairs on double-tube gauges, and the connections shifted so that the 6-ft. wall and 6-ft. longitudinal were observed with each other, and one of these two with all the others.

Tape and point-gauge readings, the standard tube, Bazin hook, and the longitudinal piezometers at the lower weir were necessarily read by different observers, and their differences involve a personal equation as well as possible inaccuracies in establishing the zeros or crest readings of the scale. The greatest care was taken in observations on the standard tube and the Bazin hook, and the work was very well done on those instruments and may be safely accepted. As to the tapes and the other apparatus, the writer does not feel quite so confident.

Table 0 consists of a series of coefficients to apply to the discharge of broad-crested and irregular weirs of a height of 11.25 ft. to give the discharge of weirs of other heights. This table is based on the effect of changing heights of sharp-crested weirs. Although the data used by the author seem to be the best we have, it is, nevertheless, to be remembered that the effect of a change of height on a broad-crested weir may be less than on a sharp edge, and hence the changes indicated by Table 0 may be too great. In other words, it seems probable that for flat-crest weirs higher than 11.25 ft., the discharge

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may be greater than that obtained in Table 0, and for similar weirs of less height than 11.25 ft., the discharge may be less than that indicated by the table, both by a small amount.

The evidence of Plate XXII, that all sharp-edge weir experiments fall substantially within 3% of a mean curve, is certainly valuable.

The curves shown on Plates XXV and XXVI are to be considered as approximate near the points where the lines deflect. There are conditions under which abrupt changes of condition arise in the discharge of weirs, but certainly nothing of the sort should appear in the standard weir at the points of deflection of its lines on these plates. More numerous experiments in the region of the deflections would show a gradual, not a sudden, change in the direction of the line.

In discussing the indication of the Francis orifice at the Lower Cornell Weir, on page 1224, the author calls attention to apparently erratic readings, and offers as an explanation the possibility of an oblique current. This orifice was ½-in. in diameter and near the center of a smooth brass plate, 12 in. long and 6 in. wide, set flush with the timber forming the side of the canal. The likelihood of a current causing the discrepancy is remote, but a much more probable explanation is that the zero of the gauge was very close to the floor, and the observer had to lie in a very cramped position to observe low heads. All readings of the scale below a head of 1 ft. are likely to be low by a small amount, including the determination of the zero, on account of the observer not getting his eye on a level with the water in the glass. The effect of this at low heads would be to give a reading ranging between 0.004 and 0.002 ft. in error, which might be either high or low depending on the relative care with which the crest reading and low-head observations were made. From Plate XXXIV it would appear that the low readings were high, which means that the zero was more accurate than the low-head readings.

The author's comparison of these indications by the approximate straight lines is open to some criticism, as the deviation of the line at some points from the true locus may be greater than that of the observations it is intended to harmonize.

On page 1265 attention is called to a note in the Laboratory Record, that the 6-ft. tape at the upper weir was below the beginning of the surface slope, and, therefore, was reading low. The statement is correct, and all the 6-ft. piezometers were similarly affected, and the fact stated, that the standard tube, 26 ft. up stream from the weir, was giving a lower reading than these, is to be explained by the relatively higher velocity past its orifices than past those near the wall. The reduction of pressure due to velocity past openings was imperfectly appreciated by the writer until experiments in a closed pipe showed that the pressure indicated by an orifice in a surface

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In the writer's reduction of the experiments at Cornell, the results of which appear in the volume of Hydraulic Tables, of which he is a joint author,* the various discharge curves were compared with each other, with the side-light of his personal knowledge of the peculiarities and characteristics of the several observers and apparatus involved. No attempt was made to establish the actual discharge in any case, but only to establish a ratio by which the discharge of a similarly dimensioned sharp-edged weir should be multiplied to give the discharge of the irregular weir. For himself, it involves less labor and conduces to accuracy to take the discharge of a sharp-edged weir from a reliable table and increase it by a known percentage than to compute the discharge *de novo* by using a special coefficient. If others prefer different methods, he has no objection, always bearing in mind, however, that the base discharge of the sharp-edged weir may be somewhat in doubt for cases beyond the range of experiment.

To the preparation of this paper the author has devoted a great amount of time and labor, a fact which no one more fully appreciates than the writer. He has assembled and presented in new form much valuable information, and for this is entitled to the thanks of all interested in the subject.

Mr. Lyman. RICHARD R. LYMAN, ASSOC. M. AM. SOC. C. E. (by letter).—Further thought and study, and the matter presented in the discussions on this paper, have confirmed the writer's conviction that the weir without end contractions is the weir which, in actual practice, will measure water in the most satisfactory way. When it is used, the diagrams† and tables presented in this paper give results, without computation, in accurate accord with existing experimental data. They give results, too, for a great variety of weirs, covering a wide range of heads, without that "troublesome correction" for velocity of approach.

Table Giving Discharges for all Weirs Without End Contractions, Within the Limits of Experimental Data.—For some purposes, tables giving discharges are more convenient than diagrams, therefore, discharges for a series of definite heads are presented in Table 69. Discharges for heads between those given may be found by interpolation.

Table Based on Formulas, Giving Discharges for High Heads.—A table of discharges, for heads higher than those for which accurate experimental data exist, by Gardner S. Williams, M. Am. Soc. C. E., has been published.‡ The results are identical with those found by the use of Plate XXIII. Therefore, no table for these discharges is presented in this paper.

* "Hydraulic Tables", Williams and Hazen, John Wiley & Sons, New York City.

† Blueprints (on cloth) of the original diagrams, and of diagrams more closely ruled, are made for field use.

‡ "American Civil Engineers' Pocket Book", pp. 867–868.

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TABLE 69.—DISCHARGES OVER SHARP-CRESTED WEIRS OF VARIOUS HEIGHTS WITHOUT END CONTRACTIONS.

These Figures were Taken from Plate XXI, and are Therefore Based on the Experiments Made by Bazin, Francis, Fteley and Stearns, and Also on Those Made in the Hydraulic Laboratories of Cornell University and the University of Utah. The Limits Included in the Diagram and the Table are Fully Covered by the Original Experiments.

Head, in inches.	Head, in feet.	Weir 0.5 ft. high.	Weir 0.75 ft. high.	Weir 1.00 ft. high.	Weir 1.50 ft. high.	Weir 2.00 ft. high.	Weir 3.00 ft. high.	Weir 4.00 ft. high.	Weir 6.00 ft. high.
2⅜	0.200	0.315	0.314	0.313	0.312	0.311	0.310	0.309	
2 7/16	0.205	0.327	0.326	0.325	0.324	0.323	0.322	0.321	
2½	0.210	0.340	0.337	0.336	0.335	0.334	0.333	0.332	
2 9/16	0.215	0.352	0.351	0.350	0.348	0.347	0.346	0.346	
2⅝	0.220	0.365	0.363	0.360	0.359	0.357	0.356	0.355	
2 11/16	0.225	0.377	0.375	0.372	0.370	0.369	0.368	0.367	
2¾	0.230	0.392	0.388	0.385	0.383	0.382	0.381	0.380	
2 13/16	0.235	0.404	0.400	0.398	0.396	0.394	0.393	0.392	
2⅞	0.240	0.420	0.415	0.412	0.408	0.406	0.405	0.404	
2 15/16	0.245	0.433	0.427	0.425	0.422	0.420	0.417	0.416	
3	0.250	0.446	0.442	0.438	0.435	0.434	0.432	0.430	
3 1/16	0.255	0.460	0.453	0.450	0.447	0.445	0.443	0.442	
3⅛	0.260	0.475	0.468	0.465	0.460	0.458	0.456	0.455	
3 3/16	0.265	0.490	0.483	0.478	0.475	0.473	0.470	0.468	
3¼	0.270	0.503	0.497	0.493	0.488	0.486	0.484	0.483	
3 5/16	0.275	0.515	0.508	0.505	0.501	0.498	0.496	0.495	
3⅜	0.280	0.530	0.524	0.518	0.514	0.510	0.507	0.506	
3 7/16	0.285	0.546	0.537	0.532	0.526	0.523	0.520	0.517	
3½	0.290	0.560	0.552	0.547	0.544	0.540	0.535	0.533	
3 9/16	0.295	0.576	0.566	0.560	0.555	0.552	0.548	0.546	
3⅝	0.300	0.595	0.584	0.576	0.570	0.566	0.563	0.560	
3⅝	0.305	0.610	0.595	0.588	0.582	0.577	0.575	0.572	
3 11/16	0.310	0.625	0.612	0.605	0.598	0.595	0.590	0.586	
3¾	0.315	0.640	0.627	0.620	0.613	0.608	0.605	0.602	
3 13/16	3.020	0.655	0.645	0.636	0.630	0.625	0.620	0.617	
3⅞	0.325	0.670	0.655	0.650	0.641	0.636	0.632	0.630	
3 15/16	0.330	0.690	0.672	0.665	0.656	0.652	0.647	0.645	
4	0.335	0.705	0.690	0.680	0.670	0.665	0.660	0.657	
4 1/16	0.340	0.720	0.705	0.697	0.688	0.683	0.675	0.673	
4⅛	0.345	0.738	0.720	0.710	0.703	0.696	0.692	0.687	
4 3/16	0.350	0.755	0.735	0.726	0.717	0.712	0.705	0.702	
4¼	0.355	0.770	0.752	0.743	0.732	0.725	0.720	0.717	
4 5/16	0.360	0.790	0.772	0.760	0.750	0.745	0.737	0.733	
4⅜	0.365	0.805	0.786	0.775	0.764	0.757	0.750	0.746	
4 7/16	0.370	0.824	0.802	0.792	0.780	0.775	0.766	0.762	
4½	0.375	0.840	0.817	0.805	0.795	0.790	0.782	0.777	
4 9/16	0.380	0.860	0.836	0.825	0.813	0.805	0.798	0.795	
4⅝	0.385	0.875	0.853	0.840	0.826	0.820	0.810	0.806	
4 11/16	0.390	0.896	0.870	0.857	0.845	0.837	0.830	0.825	
4¾	0.395	0.910	0.885	0.870	0.860	0.852	0.845	0.838	
4 13/16	0.400	0.930	0.905	0.893	0.875	0.870	0.860	0.855	0.850
4⅞	0.405	0.950	0.922	0.910	0.895	0.885	0.875	0.870	0.860
4 15/16	0.410	0.970	0.940	0.925	0.910	0.903	0.895	0.885	0.876
5	0.415	0.990	0.956	0.943	0.925	0.917	0.908	0.903	0.895
5 1/16	0.420	1.005	0.975	0.958	0.943	0.935	0.924	0.917	0.910
5⅛	0.425	1.020	0.995	0.977	0.963	0.952	0.942	0.935	0.926
5⅛	0.430	1.045	1.010	0.996	0.980	0.970	0.957	0.952	0.945
5 3/16	0.435	1.065	1.030	1.010	0.996	0.986	0.975	0.970	0.960
5¼	0.440	1.083	1.045	1.026	1.010	1.000	0.992	0.985	0.976
5 5/16	0.445	1.100	1.063	1.045	1.026	1.015	1.005	1.000	0.994
5⅜	0.450	1.120	1.080	1.060	1.040	1.030	1.015	1.010	1.030
5 7/16	0.455	1.140	1.100	1.080	1.057	1.047	0.035	1.023	1.016
5½	0.460	1.164	1.125	1.105	1.065	1.074	1.056	1.050	1.043

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TABLE 69.—(*Continued.*)

Head, in inches.	Head, in feet.	Weir 0.5 ft. high.	Weir 0.75 ft. high.	Weir 1.00 ft. high.	Weir 1.50 ft. high.	Weir 2.00 ft. high.	Weir 3.00 ft. high.	Weir 4.00 ft. high.	Weir 6.00 ft. high.
5 9/16	0.465	1.185	1.140	1.120	1.100	1.090	1.075	1.067	1.057
5 5/8	0.470	1.205	1.163	1.143	1.120	1.106	1.095	1.085	1.077
5 11/16	0.475	1.230	1.185	1.162	1.140	1.125	1.110	1.105	1.096
5 3/4	0.480	1.250	1.205	1.185	1.160	1.150	1.133	1.125	1.115
5 13/16	0.485	1.270	1.223	1.200	1.175	1.163	1.150	1.140	1.130
5 7/8	0.490	1.290	1.245	1.220	1.200	1.183	1.166	1.160	1.150
5 5/16	0.495	1.310	1.265	1.233	1.215	1.200	1.186	1.176	1.166
6	0.500	1.335	1.285	1.263	1.235	1.220	1.203	1.195	1.185
6 1/16	0.505	1.355	1.300	1.280	1.250	1.236	1.220	1.210	1.202
6 1/8	0.510	1.370	1.320	1.296	1.270	1.257	1.237	1.225	1.220
6 3/16	0.515	1.390	1.340	1.317	1.287	1.274	1.255	1.244	1.235
6 1/4	0.520	1.415	1.360	1.335	1.305	1.290	1.273	1.260	1.252
6 5/16	0.525	1.440	1.380	1.355	1.325	1.310	1.290	1.280	1.274
6 3/8	0.530	1.465	1.405	1.375	1.346	1.330	1.310	1.300	1.293
6 7/16	0.535	1.490	1.425	1.400	1.365	1.353	1.335	1.320	1.310
6 1/2	0.540	1.510	1.440	1.415	1.385	1.365	1.350	1.336	1.327
6 9/16	0.545	1.530	1.465	1.435	1.403	1.385	1.365	1.355	1.345
6 9/16	0.550	1.555	1.490	1.460	1.425	1.405	1.385	1.370	1.365
6 5/8	0.555	1.575	1.505	1.475	1.440	1.420	1.400	1.390	1.380
6 11/16	0.560	1.595	1.525	1.495	1.460	1.435	1.415	1.405	1.395
6 3/4	0.565	1.616	1.545	1.515	1.475	1.455	1.435	1.420	1.410
6 13/16	0.570	1.640	1.570	1.535	1.500	1.475	1.455	1.440	1.430
6 7/8	0.575	1.665	1.590	1.555	1.517	1.500	1.475	1.460	1.450
6 15/16	0.580	1.686	1.610	1.576	1.537	1.517	1.495	1.480	1.470
7	0.585	1.713	1.635	1.605	1.565	1.540	1.520	1.505	1.495
7 1/16	0.590	1.740	1.670	1.630	1.590	1.570	1.545	1.530	1.523
7 1/8	0.595	1.760	1.685	1.650	1.605	1.585	1.560	1.543	1.535
7 3/16	0.600	1.790	1.700	1.675	1.625	1.605	1.580	1.565	1.555
7 1/4	0.605	1.805	1.730	1.695	1.655	1.627	1.605	1.590	1.580
7 5/16	0.610	1.830	1.750	1.715	1.675	1.650	1.625	1.610	1.600
7 3/8	0.615	1.855	1.775	1.735	1.695	1.675	1.650	1.630	1.620
7 7/16	0.620	1.880	1.795	1.760	1.710	1.690	1.670	1.650	1.640
7 1/2	0.625	1.905	1.815	1.780	1.730	1.705	1.685	1.670	1.665
7 9/16	0.630	1.930	1.845	1.805	1.760	1.730	1.705	1.694	1.687
7 5/8	0.635	1.955	1.875	1.835	1.785	1.760	1.725	1.710	1.700
7 11/16	0.640	1.980	1.900	1.860	1.815	1.790	1.760	1.740	1.730
7 3/4	0.645	2.010	1.915	1.870	1.820	1.800	1.770	1.750	1.740
7 13/16	0.650	2.035	1.930	1.890	1.840	1.810	1.780	1.760	1.750
7 7/8	0.655	2.060	1.960	1.915	1.860	1.830	1.805	1.785	1.775
7 15/16	0.660	2.085	1.985	1.945	1.890	1.865	1.830	1.815	1.805
8	0.665	2.110	2.005	1.965	1.910	1.880	1.850	1.830	1.820
8 1/16	0.670	2.135	2.025	1.980	1.930	1.900	1.870	1.850	1.840
8 1/16	0.675	2.160	2.055	2.000	1.945	1.910	1.880	1.860	1.850
8 1/8	0.680	2.185	2.075	2.030	1.980	1.945	1.910	1.895	1.885
8 3/16	0.685	2.210	2.095	2.050	1.990	1.960	1.925	1.905	1.895
8 1/4	0.690	2.240	2.125	2.075	2.025	1.990	1.960	1.935	1.925
8 5/16	0.695	2.260	2.150	2.095	2.040	2.005	1.970	1.945	1.930
8 3/8	0.700	2.295	2.180	2.130	2.070	2.030	1.995	1.975	1.965
8 7/16	0.705	2.325	2.200	2.155	2.100	2.065	2.025	2.000	1.985
8 1/2	0.710	2.350	2.220	2.170	2.115	2.085	2.040	2.020	2.005
8 9/16	0.715	2.380	2.250	2.195	2.140	2.105	2.060	2.035	2.025
8 5/8	0.720	2.410	2.275	2.220	2.160	2.125	2.085	2.060	2.045
8 11/16	0.725	2.435	2.300	2.245	2.180	2.155	2.115	2.090	2.080
8 3/4	0.730	2.465	2.325	2.270	2.200	2.175	2.135	2.110	2.095
8 13/16	0.735	2.490	2.350	2.295	2.230	2.190	2.150	2.130	2.120
8 7/8	0.740	2.520	2.375	2.320	2.250	2.210	2.170	2.140	2.130
8 15/16	0.745	2.550	2.405	2.340	2.275	2.235	2.200	2.170	2.160
9	0.750	2.585	2.430	2.375	2.300	2.260	2.225	2.190	2.180
9 1/16	0.755	2.605	2.455	2.400	2.325	2.285	2.245	2.220	2.200
9 1/8	0.760	2.640	2.480	2.415	2.340	2.300	2.270	2.240	2.230
9 3/16	0.765	2.670	2.510	2.440	2.370	2.320	2.300	2.265	2.255
9 1/4	0.770	2.700	2.540	2.470	2.400	2.350	2.320	2.285	2.275
9 5/16	0.775	2.730	2.560	2.500	2.420	2.375	2.330	2.310	2.300
9 3/8	0.780	2.760	2.590	2.515	2.440	2.400	2.345	2.330	2.325
9 7/16	0.785	2.790	2.610	2.550	2.460	2.415	2.365	2.345	2.335
9 1/2	0.790	2.820	2.630	2.570	2.480	2.430	2.380	2.360	2.350
9 9/16	0.795	2.850	2.660	2.595	2.510	2.460	2.410	2.380	2.365

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TABLE 69.—(*Continued.*)

Head, in inches.	Head, in feet.	Weir 0.5 ft. high.	Weir 0.75 ft. high.	Weir 1.00 ft. high.	Weir 1.50 ft. high.	Weir 2.00 ft. high.	Weir 3.00 ft. high.	Weir 4.00 ft. high.	Weir 6.00 ft. high.
9 9/16	0.800	2.890	2.700	2.625	2.550	2.500	2.440	2.410	2.400
9 5/8	0.805	2.910	2.730	2.660	2.575	2.520	2.465	2.425	2.410
9 11/16	0.810	2.940	2.755	2.680	2.595	2.545	2.485	2.445	2.425
9 3/4	0.815	2.975	2.780	2.700	2.610	2.565	2.505	2.460	2.440
9 13/16	0.820	3.010	2.810	2.735	2.640	2.590	2.530	2.500	2.480
9 7/8	0.825	3.045	2.840	2.770	2.670	2.610	2.560	2.530	2.510
9 15/16	0.830	3.070	2.870	2.790	2.700	2.640	2.580	2.550	2.535
10	0.835	3.100	2.905	2.830	2.730	2.675	2.610	2.580	2.565
10 1/16	0.840	3.130	2.930	2.840	2.760	2.695	2.630	2.600	2.590
10 1/8	0.845	3.160	2.950	2.880	2.785	2.730	2.650	2.615	2.605
10 3/16	0.850	3.190	2.990	2.910	2.800	2.750	2.680	2.650	2.630
10 1/4	0.855	3.230	3.015	2.930	2.840	2.780	2.710	2.670	2.650
10 5/16	0.860	3.260	3.040	2.960	2.860	2.800	2.735	2.700	2.680
10 3/8	0.865	3.290	3.070	2.980	2.880	2.815	2.750	2.715	2.695
10 7/16	0.870	3.320	3.100	3.010	2.910	2.840	2.780	2.740	2.720
10 1/2	0.875	3.350	3.120	3.035	2.930	2.870	2.795	2.765	2.750
10 9/16	0.880	3.395	3.160	3.070	2.965	2.900	2.820	2.790	2.780
10 5/8	0.885	3.415	3.180	3.090	2.980	2.920	2.840	2.810	2.790
10 11/16	0.890	3.445	3.200	3.120	3.010	2.940	2.860	2.825	2.820
10 3/4	0.895	3.480	3.235	3.150	3.040	2.970	2.895	2.860	2.845
10 13/16	0.900	3.520	3.270	3.180	3.070	3.000	2.920	2.890	2.870
10 7/8	0.905	3.550	3.300	3.210	3.100	3.035	2.940	2.910	2.890
10 15/16	0.910	3.580	3.330	3.235	3.120	3.055	2.970	2.930	2.910
11	0.915	3.620	3.360	3.260	3.155	3.085	3.000	2.955	2.935
11 1/16	0.920	3.655	3.390	3.290	3.180	3.110	3.030	2.980	2.960
11 1/8	0.925	3.690	3.420	3.325	3.210	3.140	3.055	3.010	2.990
11 1/8	0.930	3.720	3.445	3.350	3.230	3.160	3.075	3.030	3.010
11 3/16	0.935	3.760	3.480	3.380	3.250	3.180	3.100	3.060	3.040
11 1/4	0.940	3.800	3.510	3.405	3.290	3.210	3.130	3.080	3.060
11 5/16	0.945	3.830	3.540	3.430	3.315	3.240	3.150	3.110	3.090
11 3/8	0.950	3.870	3.580	3.470	3.350	3.260	3.180	3.140	3.120
11 7/16	0.955	3.900	3.610	3.500	3.380	3.295	3.200	3.165	3.140
11 1/2	0.960	3.940	3.640	3.540	3.400	3.325	3.235	3.190	3.170
11 9/16	0.965	3.980	3.680	3.570	3.430	3.355	3.260	3.210	3.190
11 5/8	0.970	4.010	3.700	3.590	3.450	3.370	3.275	3.235	3.200
11 11/16	0.975	4.040	3.740	3.625	3.490	3.405	3.310	3.270	3.250
11 3/4	0.980	4.080	3.770	3.650	3.520	3.430	3.330	3.290	3.270
11 13/16	0.985	4.120	3.800	3.690	3.555	3.460	3.365	3.320	3.300
11 7/8	0.990	4.150	3.830	3.710	3.580	3.480	3.380	3.340	3.320
11 15/16	0.995	4.180	3.850	3.730	3.590	3.510	3.400	3.360	3.330
12	1.000	4.230	3.900	3.780	3.640	3.555	3.440	3.400	3.375
12 1/8	1.010	4.300	3.970	3.840	3.710	3.600	3.500	3.450	3.420
12 1/4	1.020	4.380	4.030	3.900	3.760	3.670	3.560	3.500	3.480
12 3/8	1.030	4.450	4.100	3.970	3.820	3.720	3.600	3.560	3.540
12 1/2	1.040	4.520	4.170	4.040	3.880	3.780	3.670	3.620	3.590
12 5/8	1.050	4.610	4.240	4.120	3.950	3.850	3.730	3.670	3.650
12 11/16	1.060	4.800	4.320	4.180	4.020	3.910	3.790	3.740	3.710
12 13/16	1.070	4.760	4.370	4.220	4.070	3.960	3.830	3.770	3.750
12 15/16	1.080	4.820	4.430	4.280	4.130	4.010	3.890	3.820	3.800
13 1/16	1.090	4.900	4.480	4.340	4.180	4.060	3.930	3.870	3.840
13 3/16	1.100	4.980	4.570	4.420	4.240	4.140	3.990	3.940	3.910
13 5/16	1.110	5.060	4.640	4.480	4.320	4.190	4.060	4.000	3.960
13 7/16	1.120	5.150	4.710	4.560	4.370	4.240	4.120	4.050	4.010
13 9/16	1.130	5.220	4.780	4.610	4.420	4.300	4.170	4.100	4.070
13 11/16	1.140	5.300	4.840	4.670	4.480	4.360	4.210	4.160	4.130
13 13/16	1.150	5.380	4.910	4.740	4.560	4.420	4.270	4.210	4.180
13 15/16	1.160	5.450	4.980	4.800	4.610	4.480	4.330	4.260	4.220
14 1/16	1.170	5.510	5.050	4.870	4.670	4.540	4.380	4.320	4.280
14 1/8	1.180	5.600	5.130	4.950	4.740	4.610	4.440	4.380	4.340
14 1/4	1.190	5.680	5.200	5.000	4.800	4.660	4.500	4.420	4.400
14 3/8	1.200	5.780	5.250	5.075	4.870	4.720	4.560	4.480	4.440
14 1/2	1.210	5.860	5.340	5.150	4.940	4.780	4.610	4.540	4.500
14 5/8	1.220	5.940	5.420	5.250	5.000	4.860	4.680	4.610	4.590
14 3/4	1.230	6.000	5.460	5.270	5.050	4.910	4.720	4.640	4.610
14 7/8	1.240	6.100	5.550	5.360	5.150	4.980	4.800	4.720	4.680
15	1.250	6.200	5.620	5.430	5.220	5.050	4.860	4.780	4.740
15 1/8	1.260	6.275	5.675	5.500	5.275	5.100	4.910	4.830	4.800

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TABLE 69.—(*Continued.*)

Head, in inches.	Head, in feet.	Weir 0.5 ft. high.	Weir 0.75 ft. high.	Weir 1.00 ft. high.	Weir 1.50 ft. high.	Weir 2.00 ft. high.	Weir 3.00 ft. high.	Weir 4.00 ft. high.	Weir 6.00 ft. high.
15¼	1.270		5.750	5.560	5.325	5.180	4.970	4.890	4.850
15⅜	1.250		5.820	5.620	5.380	5.225	5.000	4.940	4.900
15½	1.290		5.900	5.680	5.450	5.275	5.075	5.000	4.960
15⅝	1.300		5.975	5.775	5.525	5.350	5.150	5.050	5.020
15 11/16	1.310		6.060	5.850	5.600	5.425	5.225	5.130	5.080
15 13/16	1.320		6.150	5.920	5.675	5.500	5.275	5.200	5.150
15 15/16	1.330		6.200	6.000	5.730	5.550	5.350	5.250	5.220
16 1/16	1.340		6.300	6.050	5.800	5.620	5.400	5.320	5.260
16 3/16	1.350		6.375	6.130	5.875	5.675	5.460	5.370	5.320
16 5/16	1.360		6.450	6.200	5.940	5.750	5.520	5.430	5.380
16 7/16	1.370		6.505	6.300	6.000	5.820	5.580	5.500	5.450
16 9/16	1.380		6.625	6.375	6.080	5.900	5.650	5.560	5.525
16 11/16	1.390		6.700	6.450	6.150	5.960	5.725	5.625	5.575
16 13/16	1.400		6.780	6.530	6.230	6.040	5.770	5.675	5.640
16 15/16	1.410		6.860	6.620	6.320	6.100	5.850	5.760	5.700
17 1/16	1.420		6.950	6.675	6.375	6.150	5.920	5.820	5.760
17⅛	1.430		7.000	6.750	6.450	6.220	5.975	5.875	5.825
17¼	1.440		7.075	6.820	6.520	6.300	6.030	5.930	5.880
17⅜	1.450		7.150	6.900	6.600	6.360	6.100	6.000	5.950
17½	1.460		7.250	6.975	6.660	6.430	6.150	6.050	6.000
17⅝	1.470		7.330	7.050	6.740	6.500	6.220	6.120	6.060
17¾	1.480		7.400	7.130	6.800	6.508	6.300	6.175	6.125
17⅞	1.490		7.480	7.200	6.850	6.640	6.330	6.230	6.160
18	1.500		7.600	7.300	6.950	6.720	6.420	6.300	6.250
18⅛	1.510		7.660	7.360	7.020	6.775	6.500	6.360	6.300
18¼	1.520		7.750	7.450	7.100	6.850	6.550	6.450	6.360
18⅜	1.530		7.825	7.520	7.160	6.930	6.640	6.520	6.460
18½	1.540		7.900	7.600	7.230	7.000	6.680	6.575	6.500
18⅝	1.550		7.980	7.660	7.300	7.040	6.740	6.625	6.560
18 11/16	1.560		8.075	7.730	7.400	7.120	6.800	6.700	6.630
18 13/16	1.570		8.150	7.820	7.450	7.180	6.860	6.740	6.680
18 15/16	1.580		8.250	7.900	7.525	7.250	6.940	6.800	6.750
19 1/16	1.590		8.300	7.960	7.560	7.300	6.975	6.850	6.780

Table Giving Discharges for Weirs with Broad Crests.—Discharges over broad-crested weirs are given in Table 70. The matter it contains was prepared from Plate XXIV.

Suggestive Details for Weirs Recommended.—Suggestive details to be used in constructing weirs without end contractions are presented in Figs. 50, 51, and 52. It is recommended that the angle iron proposed for the crest have its end securely embedded in the walls of the channel; also that flush-gates, as shown in Figs. 50 and 51, be constructed so that the bottoms of the openings they cover will be on a level with the top of the floor of the channel of approach. If these flush-gates are of thin material, they can perhaps be located on the up-stream side of the bulkhead so that they will not affect materially the accuracy of the measurements. If supplied with lugs or iron loops, they may be pulled up and pushed down at pleasure.

Flush-Gates for Small Weirs.—For smaller weirs, the writer has used flush-gates of 1-in. lumber, with sheets of rubber gasket attached to them in such a way that they form a sort of flap or "check-valve". The pressure of the water on only one side of the gasket makes it fit close against the bottom of the channel and also around the three

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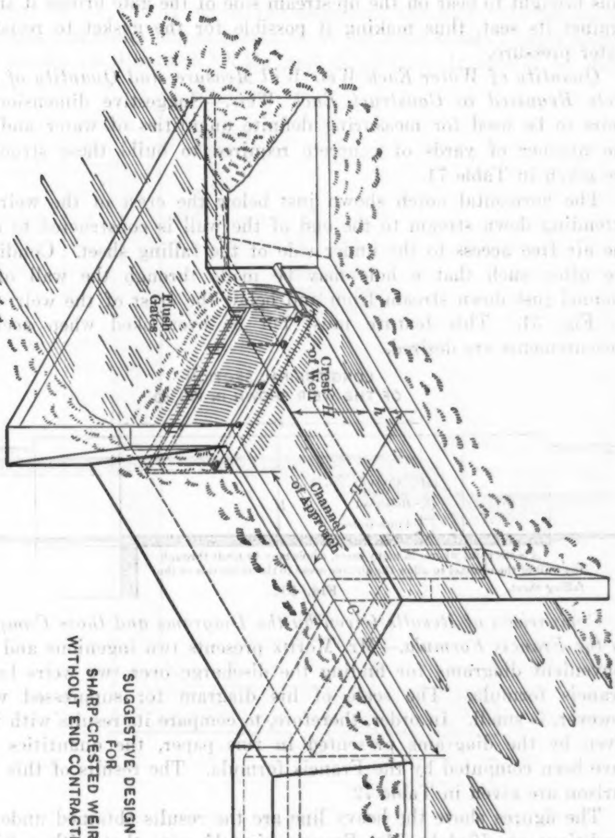

SUGGESTIVE DESIGN FOR SHARP-CRESTED WEIR WITHOUT END CONTRACTIONS

FIG. 50.

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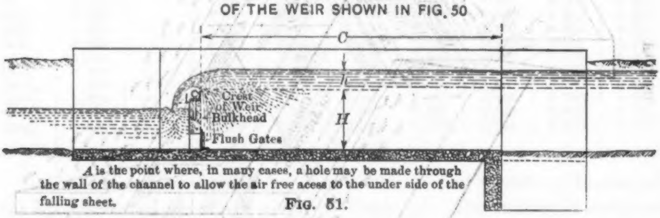
sides of the opening. A hole is cut in the rubber, and the edges of the material around this hole are fastened to the down-stream side of the gate so as to make a water-tight joint. The greater pressure thus brought to bear on the up-stream side of the gate brings it snugly against its seat, thus making it possible for the gasket to resist the water pressure.

Quantity of Water Each Weir Will Measure, and Quantity of Concrete Required to Construct Each Weir.—Suggestive dimensions of weirs to be used for measuring definite quantities of water and also the number of yards of concrete required to build these structures are given in Table 71.

The horizontal notch shown just below the crest of the weir and extending down stream to the end of the wall is constructed to allow the air free access to the under side of the falling sheet. Conditions are often such that a hole may be made through the wall of the channel just down stream from and below the crest of the weir, as at *A*, Fig. 51. This feature must not be overlooked when accurate measurements are desired.

LONGITUDINAL SECTION
OF THE WEIR SHOWN IN FIG. 50



A is the point where, in many cases, a hole may be made through the wall of the channel to allow the air free acess to the under side of the falling sheet.

FIG. 51.

Comparison of Results Given by the Diagrams and those Computed by the Francis Formula.—Mr. Moritz presents two ingenious and very convenient diagrams for finding the discharge over two weirs by the Francis formula. The scale of his diagram for suppressed weirs, however, is small. In order, therefore, to compare its results with those given by the diagrams presented in this paper, the quantities used have been computed by the Francis formula. The results of this comparison are given in Table 72.

The figures above the heavy line are the results obtained under the conditions specified by Mr. Francis himself, *viz.*, that with a "depth of channel leading to the weir * * * as small as three times that on the weir" the Francis formula "agrees with experiment within less than one per cent.", therefore "if the canal leading to the weir has a suitable depth, it will be requisite only when great precision is required * * * to go through the troublesome calculation of correcting the depth on the weir".*

* "Lowell Hydraulic Experiments", pp. 134 and 135.

Table 72 shows that, even when the conditions specified by Mr. Francis himself are satisfied (conditions which all experienced engineers attempt to follow), differences occur, varying from 1.3 to 7.0 per cent. Plate XXII shows the degree of accuracy with which the diagrams and tables herein presented fit the great mass of experimental data shown on Plates XXVII*a* and XXVII*b*. It should be noted Mr. Lyman.

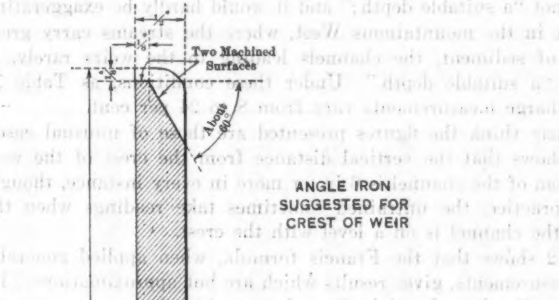


FIG. 52.

however, that the Francis data shown on Fig. 8, Plate XXVII*a*, on which the Francis formula is based, are but an exceedingly small fraction of the accepted data. If it be argued that these diagrams do not give results which are absolutely correct, it cannot be denied that these results indicate positively that one right line or formula of the Francis form, namely, $Q = mh^n$, cannot be made to fit accurately these actual experimental results.

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In the preparation of Table 72, the velocity of approach has not been given consideration, for two reasons: One is that it is truly, as Mr. Francis says, a "troublesome correction"; the other is that such corrections are rarely made in practice.

The figures below the heavy line are presented to show the greater errors given by the Francis formula if the "channel leading to the weir" has not "a suitable depth;" and it would hardly be exaggerating to say that in the mountainous West, where the streams carry great quantities of sediment, the channels leading to the weirs rarely, if ever, have "a suitable depth." Under these conditions, as Table 72 shows, discharge measurements vary from 8 to 24 per cent.

Some may think the figures presented are those of unusual cases. Table 72 shows that the vertical distance from the crest of the weir to the bottom of the channel is 6 in. or more in every instance, though, in actual practice, the untrained sometimes take readings when the bottom of the channel is on a level with the crest.

Table 72 shows that the Francis formula, when applied generally to weir measurements, gives results which are but approximations. Its use is the application of a right line where nothing but a curve can fit, and the curve that fits must be a flexible one. The diagrams on Plate XXI are flexible, and they fit accurately every case within the limits of existing experimental data.

Why Heads Were Measured 15 Ft. Up Stream.—Although the heads considered in this paper were measured generally 15 ft. up stream from the crest by means of a "tape", this comparatively great distance was selected and used as a "standard", at the Hydraulic Laboratory of Cornell University. Very high as well as very low heads had to be measured with this same tape, and it was feared that at a less distance the "curved surface over the crest" might affect the readings for high heads. The cross-section of the channel of approach was so great, and the floor in it so nearly horizontal, that the results thus obtained are perhaps the same as if readings had been taken as Mr. Hazen suggests, at a distance up stream "equal to 2.5 times the height of the weir." However, unless some limit fixes the maximum allowable head on a weir of given height, perhaps it would be better to have the heads measured up stream from the crest a distance not less than some multiple of the maximum head to be allowed on the weir. Had the rule Mr. Hazen suggests been applied in the case of the upper standard weir in the Hydraulic Laboratory of Cornell University, the "tape" readings would have been taken up stream from the crest of the weir a distance of about 28 ft., because the weir is 11.25 ft. high.

Weirs Having Dimensions Proportional.—Concerning one other of the many excellent matters presented in the discussion by Mr. Hazen, *viz.,* "that weirs which, in all respects, are proportional to each other

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TABLE 70.—DISCHARGE, IN CUBIC FEET PER SECOND PER FOOT OF LENGTH, OVER BROAD-CRESTED WEIRS WITHOUT END CONTRACTIONS.

These Quantities Were Read from Plate XXIV, Which Is Based on Experiments Made on Weirs 11.25 Ft. High in the Hydraulic Laboratory of Cornell University. For the Discharge over Weirs of Heights Other than 11.25 Ft., Apply the Coefficients Given in Table 0.

Head, in inches.	Head, in feet.	Weir 6 in. wide.	Weir 1 ft. 0 in. wide.	Weir 1 ft. 6 in. wide.	Weir 2 ft. 0 in. wide.	Weir 2 ft. 6 in. wide.	Weir 3 ft. 0 in. wide.	Weir 3 ft. 6 in. wide.
1 9/16	0.13	0.134	0.134	0.134	0.134	0.134	0.134	0.134
1 11/16	0.14	0.149	0.149	0.149	0.149	0.149	0.149	0.149
1 13/16	0.15	0.165	0.165	0.165	0.165	0.165	0.165	0.165
1 15/16	0.16	0.181	0.181	0.181	0.181	0.181	0.181	0.181
2 1/16	0.17	0.197	0.197	0.197	0.197	0.197	0.197	0.197
2 3/16	0.18	0.214	0.214	0.214	0.214	0.214	0.214	0.214
2 1/4	0.19	0.232	0.232	0.232	0.232	0.232	0.232	0.232
2 3/8	0.20	0.250	0.250	0.250	0.250	0.250	0.250	0.250
2 1/2	0.21	0.268	0.268	0.268	0.268	0.268	0.268	0.268
2 5/8	0.22	0.288	0.288	0.288	0.288	0.288	0.288	0.288
2 3/4	0.23	0.305	0.305	0.305	0.305	0.305	0.305	0.305
2 7/8	0.24	0.326	0.326	0.326	0.326	0.326	0.326	0.326
3	0.25	0.346	0.346	0.346	0.346	0.346	0.346	0.346
3 1/8	0.26	0.366	0.366	0.366	0.366	0.366	0.366	0.366
3 1/4	0.27	0.386	0.386	0.386	0.386	0.386	0.386	0.386
3 3/8	0.28	0.405	0.405	0.405	0.405	0.405	0.405	0.405
3 1/2	0.29	0.430	0.430	0.430	0.430	0.430	0.430	0.430
3 5/8	0.30	0.450	0.450	0.450	0.450	0.450	0.450	0.450
3 13/16	0.32	0.495	0.495	0.495	0.495	0.495	0.495	0.495
4 1/16	0.34	0.540	0.540	0.540	0.540	0.540	0.540	0.540
4 5/16	0.36	0.590	0.590	0.590	0.590	0.590	0.590	0.590
4 9/16	0.38	0.645	0.640	0.640	0.640	0.640	0.640	0.640
4 13/16	0.40	0.705	0.685	0.685	0.685	0.685	0.685	0.685
5 1/16	0.42	0.765	0.735	0.735	0.735	0.735	0.735	0.735
5 1/4	0.44	0.830	0.785	0.785	0.785	0.785	0.785	0.785
5 1/2	0.46	0.900	0.840	0.840	0.840	0.840	0.840	0.840
5 3/4	0.48	0.970	0.895	0.895	0.895	0.895	0.895	0.895
6	0.50	1.040	0.950	0.950	0.950	0.950	0.950	0.950
6 5/8	0.55	1.230	1.115	1.095	1.095	1.095	1.095	1.095
7 3/16	0.60	1.425	1.285	1.245	1.245	1.245	1.245	1.245
7 13/16	0.65	1.645	1.470	1.400	1.400	1.400	1.400	1.400
8 3/8	0.70	1.875	1.665	1.570	1.570	1.570	1.570	1.570
9	0.75	2.11	1.865	1.735	1.735	1.735	1.735	1.735
9 5/8	0.80	2.36	2.08	1.935	1.915	1.915	1.915	1.915
10 3/16	0.85	2.61	2.30	2.14	2.09	2.09	2.09	2.09
10 13/16	0.90	2.87	2.52	2.36	2.28	2.28	2.28	2.28
11 3/8	0.95	3.11	2.76	2.57	2.47	2.47	2.47	2.47
12	1.00	3.37	3.00	2.80	2.66	2.66	2.66	2.66
12 5/8	1.05	3.62	3.26	3.04	2.86	2.86	2.86	2.86
13 3/16	1.10	3.88	3.53	3.30	3.10	3.07	3.07	3.07
13 13/16	1.15	4.16	3.80	3.56	3.35	3.29	3.29	3.29
14 3/8	1.20	4.44	4.10	3.83	3.60	3.51	3.51	3.51
15	1.25	4.70	4.38	4.10	3.85	3.72	3.72	3.72
15 5/8	1.30	5.00	4.68	4.38	4.12	3.96	3.95	3.95
16 13/16	1.40	5.58	5.28	4.94	4.66	4.48	4.40	4.40
18	1.50	6.16	5.92	5.56	5.24	5.02	4.88	4.88
19 3/16	1.60	6.76	6.54	6.16	5.82	5.58	5.42	5.36
20 3/8	1.70	7.40	7.24	6.82	6.42	6.18	6.00	5.86
21 5/8	1.80	8.04	7.94	7.50	7.08	6.78	6.58	6.42
22 13/16	1.90	8.76	8.74	8.22	7.76	7.44	7.22	7.02
24	2.00	9.42	9.42	8.96	8.44	8.08	7.84	7.64
25 3/16	2.10	10.20	10.20	9.70	9.16	8.78	8.50	8.26
26 3/8	2.20	10.95	10.95	10.50	9.90	9.50	9.18	8.94
27 5/8	2.30	11.70	11.70	11.30	10.65	10.20	9.88	9.60
28 13/16	2.40	12.50	12.50	12.10	11.45	10.95	10.60	10.30
30	2.50	13.30	13.30	12.95	12.20	11.70	11.35	11.00

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TABLE 70.—(*Continued.*)

Head, in inches.	Head, in feet.	Weir 6 in. wide.	Weir 1 ft. 0 in. wide.	Weir 1 ft. 6 in. wide.	Weir 2 ft. 0 in. wide.	Weir 2 ft. 6 in. wide.	Weir 3 ft. 0 in. wide.	Weir 3 ft. 6 in. wide.
31 3/16	2.60	14.15	14.15	13.85	13.10	12.55	12.10	11.80
32 3/8	2.70	15.00	15.00	14.75	13.95	13.35	12.90	12.55
33 5/8	2.80	15.90	15.90	15.70	14.80	14.15	13.70	13.30
34 13/16	2.90	16.75	16.75	16.60	15.70	15.00	14.50	14.05
36	3.00	17.65	17.65	17.60	16.60	15.90	15.35	14.90
38 3/8	3.20	19.50	19.50	19.50	18.50	17.70	17.05	16.55
40 13/16	3.40	21.5	21.5	21.5	20.5	19.60	18.90	18.25
43 3/16	3.60	23.5	23.5	23.5	22.6	21.6	20.8	20.1
45 5/8	3.80	25.4	25.4	25.4	24.7	23.6	22.7	22.0
48	4.00	27.6	27.6	27.6	26.9	25.8	24.8	24.0
50 3/8	4.20	29.7	29.7	29.7	29.2	27.9	26.8	25.8
52 13/16	4.40	31.8	31.8	31.8	31.5	30.0	28.9	27.8
55 3/16	4.60	34.2	34.2	34.2	34.0	32.4	31.1	30.0
57 5/8	4.80	36.6	36.6	36.6	36.5	34.9	33.5	32.2
60	5.00	38.8	38.8	38.8	38.8	37.3	35.8	34.5

TABLE 71.—SUGGESTIVE DIMENSIONS FOR WEIRS TO BE USED FOR MEASURING VARIOUS QUANTITIES OF FLOWING WATER.

Further Information, Concerning the Meaning of the Notation Used, is Given on Figs. 50 and 51.

Minimum, Q, in cubic feet.	Maximum, Q, in cubic feet.	h, in feet.	H, in feet.	Height of wall, in feet.	L, length of weir, in feet.	C, length of channel, in feet.	Thickness of floor, in inches.	Thickness of bottom of wall, in inches.	Thickness of top of wall, in inches.	Length of wing wall, in feet.	Depth of cut-off wall, in feet.	Concrete, in cubic yards.
0.05	0.25	0.5	0.5	1.0	0.2	2	6	8	6	2.0	1.0	0.5
0.15	0.60	0.5	0.5	1.0	0.5	4	6	8	6	2.0	1.0	0.7
0.30	1.30	0.5	0.5	1.0	1.0	6	6	8	6	2.0	1.0	0.9
0.45	1.90	0.5	0.75	1.25	1.5	8	6	8	6	2.0	1.0	1.4
0.60	2.50	0.5	0.75	1.25	2.0	10	6	8	6	2.5	1.0	2.2
0.75	3.2	0.5	0.75	1.25	2.5	10	6	8	6	2.5	1.0	2.3
1.0	3.7	0.5	1.0	1.5	3.0	10	6	8	6	2.5	1.0	2.4
1.5	11.0	1.0	1.0	2.0	3.5	12	6	8	6	3.0	1.0	3.5
2.0	15.0	1.0	1.0	2.0	4.0	12	6	8	6	3.0	1.0	3.7
2.5	18.0	1.0	1.5	2.5	5.0	12	6	8	6	3.0	1.0	4.0
3.0	21.0	1.0	1.5	2.5	6.0	14	8	12	6	3.0	1.5	5.7
3.5	25.0	1.0	1.5	2.5	7.0	14	8	12	6	4.0	1.5	6.2
4.0	28.0	1.0	2.0	3.0	8.0	14	8	12	6	5.0	1.5	10.0
4.5	32.0	1.0	2.0	3.0	9.0	16	8	12	8	5.0	1.5	11.4
5.0	65.0	1.5	2.0	3.5	10.0	16	8	12	8	8.0	2.0	18.0
6.0	71.0	1.5	2.0	3.5	11.0	16	8	12	8	8.0	2.0	18.7
7.0	78.0	1.5	2.0	3.5	12.0	16	12	15	8	8.0	2.0	24.5
8.0	84.0	1.5	2.0	3.5	13.0	16	12	15	8	8.0	2.0	25.2
9.0	91.0	1.5	3.0	4.5	14.0	18	12	15	8	9.0	2.5	34.6
10.0	97.0	1.5	3.0	4.5	15.0	18	12	18	12	10.0	3.0	44.6
11.0	104.0	1.5	3.0	4.5	16.0	18	12	18	12	10.0	3.0	47.5
12.0	170.0	2.0	3.0	5.0	17.0	18	12	18	12	10.0	3.0	48.7
13.0	180.0	2.0	3.0	5.0	18.0	18	12	18	12	10.0	3.0	49.9
14.0	200.0	2.0	3.0	5.0	20.0	20	12	18	12	10.0	3.0	55.5
15.0	215.0	2.0	4.0	6.0	22.0	20	12	24	12	12.0	4.0	78.0
16.0	230.0	2.0	4.0	6.0	24.0	20	12	24	12	12.0	4.0	80.8
17.0	250.0	2.0	4.0	6.0	26.0	20	12	24	12	12.0	4.0	83.6
18.0	270.0	2.0	4.0	6.0	28.0	20	12	24	12	12.0	4.0	86.4
19.0	290.0	2.0	4.0	6.0	30.0	20	12	24	12	12.0	4.0	89.2
20.0	310.0	2.0	4.0	6.0	32.0	20	12	24	12	12.0	4.0	92.2

will have the same coefficients", the writer wishes to say that for years he has attempted to get into clear form this same idea. Now that this principle has been clearly stated, he will attempt to demonstrate in the Hydraulic Laboratory of the University of Utah whether or not it is correct. Mr. Lyman.

TABLE 72.—PERCENTAGE OF DIFFERENCE BETWEEN QUANTITIES OF DISCHARGE OVER WEIRS OF VARIOUS HEIGHTS, AS GIVEN BY PLATE XXI, AND AS COMPUTED BY THE FRANCIS FORMULA: $Q_F = 3.33\, H^{\frac{3}{2}}$.

Q_F = Discharge, in cubic feet per second.

H = Height of weir, in feet.

The quantities above the heavy line are within the limits for the formula specified by Mr. Francis.

Head, in feet.	Q_F	QUANTITY, AS READ FROM PLATE XXI.							
		$H = 0.5$ ft.	$H = 0.75$ ft.	$H = 1.0$ ft.	$H = 1.5$ ft.	$H = 2.0$ ft.	$H = 3.0$ ft.	$H = 4.0$ ft.	$H = 6.0$ ft.
0.2	0.298	0.315 5.4%	0.314 5.1%	0.313 4.8%	0.312 4.5%	0.311 4.2%	0.310 3.9%	0.309 3.6%	
0.3	0.547	0.595 8.1%	0.584 6.3%	0.576 5.0%	0.570 4.0%	0.566 3.4%	0.563 2.8%	0.560 2.3%	
0.4	0.842	0.930 9.5%	0.905 7.0%	0.893 5.7%	0.875 3.8%	0.870 3.2%	0.860 2.1%	0.855 1.5%	0.850 0.9%
0.7	1.950	2.295 15.0%	2.180 10.5%	2.130 8.5%	2.070 5.8%	2.030 3.9%	1.995 2.3%	1.975 1.3%	1.965 0.8%
1.0	3.330	4.230 21.3%	3.900 14.6%	3.780 11.9%	3.640 8.5%	3.555 6.3%	3.440 3.2%	3.400 2.1%	3.375 1.6%
1.3	4.936	6.51 24.2%	5.975 17.4%	5.775 14.5%	5.525 10.7%	5.350 7.7%	5.150 4.2%	5.050 2.3%	5.020 1.7%
1.6	6.739	....	8.40 19.8%	8.07 16.5%	7.69 12.4%	7.41 9.2%	7.08 4.8%	6.94 3.0%	6.89 2.3%

The Weir Without End Contractions Gives the Most Accurate Results.—Mr. Moritz says the statement that the "sharp-crested weir without end contractions" gives "the most accurate results with the smallest amount of work * * * is open to serious question", as the use of such weirs "would be practically impossible on account of the large quantity of silt carried at certain seasons of the year" in the canals of some irrigation projects.

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Great Velocity of Approach and Little Sediment Deposited.—The first of the six reasons why these weirs are best, given near the beginning of this paper, was that, with this structure, the velocity of the water above the weir is greatest, and therefore its capacity for carrying silt over the crest is also greatest. The second of the six reasons is that, with this weir, the channel of approach must be made of concrete, lumber, or some other material, that will retain a definite rectangular form; it is easy, therefore, to determine whether or not the channel is clean, and it must be clean to give good results. Another reason may now be added: This weir is best because its crest may be fixed in position permanently, and flush-gates may be built under it. Through these gates silt may pass when measurements are not actually being taken. Thus is maintained a relatively great velocity in this comparatively small channel of approach, and the channel is kept clean.

The Diagrams and Tables Fit Great Variety of Weirs and Great Variation of Heads.—Again, as previously explained, the diagrams and tables submitted in this paper (Plate XXI and Table 69) fit, and fit accurately, a great variety of weirs of this type, and do so for a comparatively large range of heads. The Francis diagram or formula for suppressed weirs, based on only 17 experiments, cannot be expected to fit, nor does it fit, the great variety of weirs and the large range of heads to which it has been and is being applied. The smallest head used by Mr. Francis was greater than 0.73 ft., and the largest less than 1.1 ft.; his experiments, therefore, cover a difference in head of but little more than 3 in. Moreover, all his experiments were made on only one weir with only one height.

Francis Formula Fits Within Very Narrow Limits.—The Francis experiments are shown on Fig. 8, Plate XXVII*a*. The results obtained by other experiments are presented on both sides of this figure. They indicate that if Francis had used higher and lower heads, he would have discovered that no one equation of the form he uses ($Q = mh^n$) can fit accurately a series of experiments on weirs of the type herein considered.

The Best Practical Measuring Device Available.—Mr. Moritz closes his argument in favor of the Cippoletti weir with these words:

"In the writer's opinion, there is no better practical device, at present available, than the Cippoletti weir for measuring the quantities encountered in the majority of irrigation laterals."

The velocity of the water approaching the Cippoletti weir is less than that of the water approaching the weir without end contractions. In order to give accurate results, the cross-section of the channel of approach for the former must be greater than that for the latter. The deposit of silt, therefore, will be greater above the Cippoletti weir than above the weir without end contractions. If, then, with the Cippoletti

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weir, as he explains, "one or two good cleanings of the pool during the season will generally suffice", less cleaning will be required for the weir without end contractions. Still more important is the fact that a conscientious water master may clean his canal so as to make a large pool, and a water master of the other sort may make but a small pool, or none at all. Table 72 shows what effect a change in the cross-section of the "pool" or channel of approach has on the discharge. In the case of the suppressed weir, the channel to be cleaned has a fixed cross-section, of concrete, timber, or other material, that will retain its definite rectangular section. With sluice-gates through the bulkhead under the crest of the weir, this channel can be cleaned with the aid of the water much more easily than can the "pool" of which Mr. Moritz speaks.

If the channel above the suppressed weir has 6 in. of silt in it, the discharge (by taking this fact into account) can be found from the diagrams and tables herewith submitted. This cannot be done with the Cippoletti weir.

The writer does not see wherein the use of the tables and diagrams recommended in this paper requires any greater care than does the use of the Cippoletti weir and the diagram Mr. Moritz submits. Neither does he concede that the Cippoletti weir is the simpler of the two, in the sense of its being easier to use; but he will concede that it may be installed at a little less cost.

Mr. Moritz does not give a reference to the experiments which he states show that the Cippoletti weir gives results "within an error of from 2 to 4% when the head is not greater than one-third of the crest length." However, as the Francis formula is used for computing the discharge over the Cippoletti weir, it is important to specify the distance of each contraction from the sides and from the bottom of the channel. Of course, the greater the cross-section of the channel of approach, the more nearly will conditions approach the ideal; and the nearer this ideal is approached, the greater will be the tendency to fill the channel with silt.

Accuracy of Results.—Mr. Moritz claims that the Cippoletti weir, under what he intimates are rather limited conditions, gives results "within an error of from 2 to 4%"; but he fails to give the quantity or value of the data on which he bases this assertion. He does explain, however, that "the quantities in these experiments were measured volumetrically", and intimates in the same sentence that there are errors in the tables and diagrams herein presented "due to the use of a calibrated measuring device * * * such as the 'standard weir' used in the Cornell Experiments," giving results which may be "accepted as accurate to within 2 or 3 per cent."

Evidently, Mr. Moritz has gone carefully over the matter presented in this paper, but he did not see for the moment that the "2 or 3 per

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cent." applies to "Broad-Crested Weirs" (page 1199), and not to "Sharp-Crested Weirs Without End Contractions" (page 1192). It is for the latter weirs and for the diagrams and tables which accompany them that this paper makes its greatest claim. The results are based on the twenty "classic" series of experiments named on the left of Plate XXII. This plate indicates clearly with what degree of accuracy the diagrams and tables of this paper fit this unparalleled array of hydraulic data.*

Discrepancies in Quantities for Weirs with Broad Crests.—To furnish the "additional light" for which Mr. Moritz asks concerning certain runs on broad-crested weirs, Fig. 53 has been made. The curves shown are drawn to fit the points based on the original experiments. These points are represented by large dots. The results reached by Mr. Williams† are shown by small crosses, and those obtained by Mr. Horton‡ are represented by small dots within triangles.

The accuracy with which the results given by the diagrams on Plates XXVI and XXXI fit those obtained by using the original data is shown by the circles on Fig. 53. This information and the original data on which Messrs. Williams and Horton also based their results are given in Table 73. From the quantities given, and their signs, in the column marked "Percentage of difference", it is seen that the results obtained by using Plates XXVI and XXXI fit the original experiments almost perfectly. There are only five individual experiments which show a difference greater than 2%, and two of these would show still smaller differences but for the fact that slight errors in calculation were made in the original computations.

Accuracy with Which Diagrams Fit Original Data.—Attention is again drawn to Plates XXX and XXXII*a*. These plates show that the diagrams giving mean values furnish results which can be used with greater confidence than can the results of the individual runs. Other plates in this paper give the same information concerning the accuracy of the other diagrams.

Although the Suppressed Weir Is Recommended as the Best Weir, Other Measuring Devices Are Necessary.—It was not, as Mr. Johnston seems to have understood, the writer's intention that the weir be adopted by legislation as the only device for measuring water. The intention was to advocate that, by legislation, the weir without end contractions be adopted as the "standard weir". It can be used to obtain accurate results wherever any other type of weir can be used, and in many places and under many conditions where reliance can be placed on no other weir. If, as Mr. Robinson says, this type of weir were made standard by Federal Statute, then it would soon be used

* Reference is given in the proper places in this paper to the original sources from which these data came.

† "American Civil Engineers' Pocket Book", p. 869.

‡ U. S. Geological Survey, Water Supply Paper No. 200, pp. 190 and 192.

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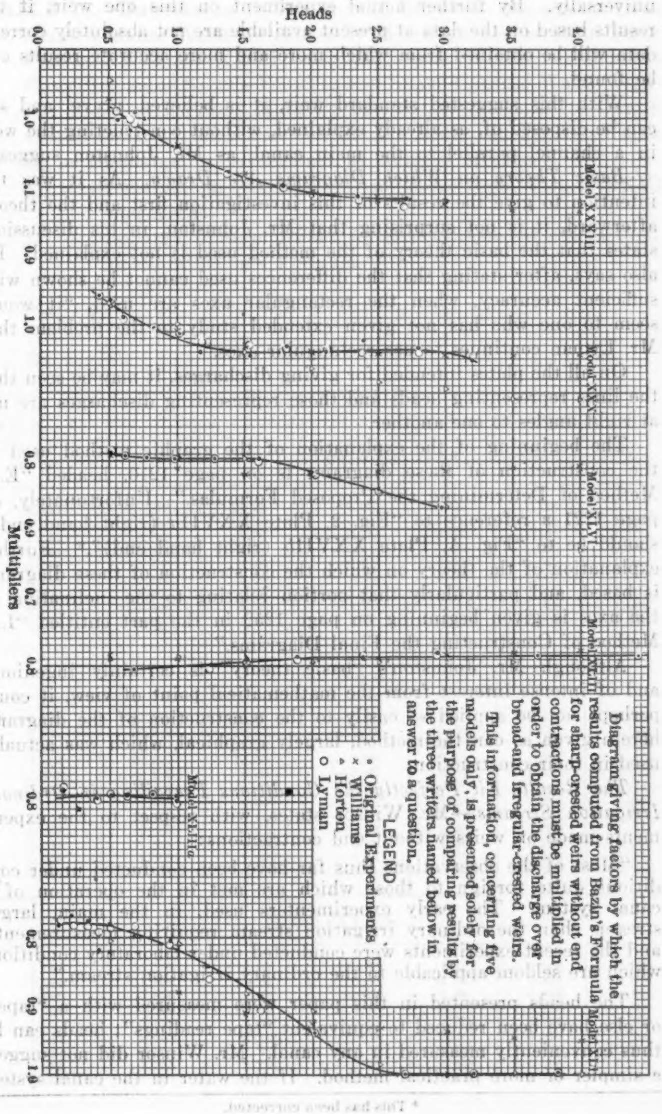

Fig. 58.

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universally. By further actual experiment on this one weir, if the results based on the data at present available are not absolutely correct, data will be obtained from which more and more accurate results can be found.

With this suggested standard weir, it is believed, gravel and silt can be disposed of, as already explained, without constructing the weir in a channel parallel to the main canal, as Mr. Johnston suggests.

Basic Theory on Which Diagrams Are Drawn.—As it was the intention to give the results of this investigation first and the theory afterward, it is not surprising that Mr. Johnston, in his discussion, states that the basic theory of the method used is not explained. He also says, after stating that the differences used cannot be shown with sufficient accuracy, when the rectangular axes are used, "it would seem to one who has not given extended study to the problem that Mr. Lyman continues to use rectangular axes."

On all the plates intended for giving discharges, it may be seen that the lines representing heads and those representing discharges are not at right angles to one another.

The beginning of the explanation of the graphic method used in the construction of these diagrams is on page 1210, headed "E.—Method of Determining the Proposed Formulas." Unfortunately, on page 1221 a reference to "Fig. 2, Plate XXVII*a* (right hand end)" should be to "Fig. A, Plate XXVII*b* (right hand end)".* Further explanation of the theory on which the construction of these diagrams is based, and particularly that portion relating to the inclination of the axes is given beginning on page 1252 in the part entitled "I.—Method of Constructing the Final Diagrams."

Although Mr. Johnston's "basic theory" is certainly ingenious, and of intense interest from the mathematical point of view, it could perhaps not be applied so easily to the construction of the diagrams herein given as can the method, largely graphical, which was actually used in their construction.

The Results Fit Perfectly the Conditions Prevailing in Ordinary Irrigation Streams.—Mr. Winsor states, with respect to the experiments made on weirs without end contractions:

"Most of the observations thus far have been conducted under conditions quite foreign to those which are met in the operation of a canal system. The early experimenters used, in the main, larger streams than the ordinary irrigation stream requiring measurements, and the recent experiments were conducted under laboratory conditions which are seldom applicable to the ordinary irrigation stream."

The heads presented in this paper were measured with a "tape", or else have been reduced to equivalent "tape readings"; heads can be thus conveniently measured in any canal. Mr. Winsor did not suggest a simpler or more practical method. If the water in the canal system

* This has been corrected.

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TABLE 73.—DIFFERENCES IN PERCENTAGE BETWEEN DISCHARGES OVER BROAD-CRESTED WEIRS AS ACTUALLY MEASURED AND AS READ FROM PLATES XXVI AND XXXI; ALSO FACTORS BY WHICH THE RESULTS COMPUTED FROM BAZIN'S FORMULA FOR SHARP-CRESTED WEIRS WITHOUT END CONTRACTIONS MUST BE MULTIPLIED IN ORDER TO OBTAIN THE DISCHARGE OVER BROAD-CRESTED AND IRREGULAR-CRESTED WEIRS.

Q_m = Discharge, in Second-Feet, as Actually Measured.

Q_L = Discharge, in Second-Feet, as Read from Plates XXVI and XXXI.

Q_B = Discharge, in Second-Feet, for Sharp-Crested Weirs, as Computed by Bazin's Formula.

$$M_m = \frac{Q_m}{Q_B}.$$

$$M_L = \frac{Q_L}{Q_B}.$$

MODEL XXXIII.

Head, in feet.	Q_m	Q_L	Percentage of difference.	Q_B	M_m	M_L
2.660	16.272	16.45	−1.09	14.55	1.118	1.130
2.352	13.464	13.55	−0.64	12.07	1.116	1.123
1.784	8.747	8.76	−0.15	7.94	1.101	1.102
1.394	5.914	5.94	−0.44	5.475	1.079	1.085
0.992	3.439	3.460	−0.61	3.300	1.042	1.048
0.752	2.197	2.240	−1.96	2.192	1.001	1.022
0.679	1.866	1.896	−1.61	1.882	0.992	1.007
0.653	1.808	1.785	+1.27	1.775	1.018	1.004
0.553	1.397	1.375	+1.57	1.389	1.006	0.992

MODEL XXX.

Head, in feet.	Q_m	Q_L	Percentage of difference.	Q_B	M_m	M_L
3.187	20.152	20.00	+0.75	19.17	1.051	1.043
2.790	16.219	16.42	−1.24	15.65	1.037	1.049
2.384	12.631	12.88	−1.97	12.35	1.022	1.042
2.094	10.577	10.55	−0.26	10.15	1.041	1.039
1.793	8.282	8.30	−0.22	8.00	1.034	1.037
1.532	6.607	6.504	+1.56	6.317	1.045	1.030
1.174	4.399	4.350	+1.11	4.238	1.039	1.027
1.010	3.436	3.430	+0.18	3.390	1.012	1.011
0.802	2.409	2.416	−0.29	2.409	1.000	1.002
0.613	1.608	1.600	+0.48	1.621	0.992	0.997
0.508	1.195	1.200	−0.42	1.239	0.966	0.970
0.427	0.921	0.920	+0.11	0.960	0.959	0.958

MODEL XLVI.

Head, in feet.	Q_m	Q_L	Percentage of difference.	Q_B	M_m	M_L
2.965	14.901	14.85	+0.34	17.18	0.868	0.865
2.486	11.032	11.13	−0.89	13.13	0.841	0.848
2.030	7.895	7.95	−0.70	9.68	0.816	0.822
1.597	5.360	5.38	−0.37	6.711	0.798	0.803
1.232	3.628	8.624	+0.11	4.556	0.797	0.799
0.972	2.549	2.549	±0.00	3.201	0.797	0.798
0.784	1.856	1.857	−0.05	2.328	0.797	0.797
0.602	1.255	1.256	−0.08	1.578	0.796	0.796
0.503	0.967	0.963	+0.41	1.222	0.791	0.788

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TABLE 73.—(*Continued.*)

MODEL XLIII.

Head, in feet.	Q_m	Q_L	Percentage of difference.	Q_B	M_m	M_L
4.432	25.011	24.94	+0.28	31.95	0.783	0.781
3.661	18.651	18.69	−0.21	23.78	0.785	0.786
2.935	13.200	13.27	−0.53	16.94	0.779	0.783
2.360	9.608	9.54	+0.71	12.15	0.791	0.784
1.890	6.841	6.87	−0.42	8.66	0.789	0.794
1.480	4.767	4.78	−0.06	6.012	0.793	0.796
1.206	3.520	3.550	−0.85	4.413	0.798	0.805
0.689	1.544	1.545	−0.07	1.924	0.803	0.804

MODEL XLIII*a*.

Head, in feet.	Q_m	Q_L	Percentage of difference.	Q_B	M_m	M_L
0.986	2.618	2.625	−0.27	3.270	0.800	0.803
0.981	2.568	2.605	−1.44	3.245	0.791	0.803
0.786	1.847	1.850	−0.16	2.337	0.791	0.793
0.621	1.309	1.310	−0.08	1.653	0.792	0.793
0.496	0.945	0.942	+0.32	1.196	0.790	0.788
0.392	0.671	0.674	−0.45	0.844	0.795	0.798
0.279	0.421	0.409	+2.85	0.520	0.810	0.787
0.161	0.184	0.185	−1.09	0.238	0.774	0.778

MODEL XLI.

Head, in feet.	Q_m	Q_L	Percentage of difference.	Q_B	M_m	M_L
3.841	25.531	25.47	+0.24	25.59	0.999	0.999
3.176	18.995	19.04	−0.24	19.06	0.998	0.999
2.674	14.624	14.36	+1.80	14.66	0.999	0.999
2.021	9.021	9.00	+0.23	9.61	0.938	0.937
1.601	5.936	6.08	−2.42	6.72	0.883	0.905
1.233	3.835	3.920	−2.21	4.562	0.841	0.860
0.941	2.476	2.485	−0.36	3.058	0.810	0.814
0.670	1.472	1.442	+2.04	1.845	0.798	0.782
0.488	0.911	0.917	−0.66	1.167	0.781	0.786
0.417	0.759	0.735	+3.16	0.926	0.819	0.794
0.330	0.520	0.526	−1.15	0.669	0.777	0.787
0.210	0.273	0.278	−1.83	0.355	0.769	0.783
0.122	0.130	0.128	+1.54	0.166	0.784	0.772

is measured with any degree of care, then, with the heads measured as above, the conditions under which these experiments were made appear to be identical with those "met in the operation of a canal system."

We may wait long for many series of experiments to be made with such care that they will deserve a place among those used in this paper. It will certainly be a much longer time before future experiments will warrant disregarding these, even if the results they give are not "absolutely correct."

As to the quantity of water actually measured: Plate XXII gives the list of the experiments on which Plate XXI is based. For the results given by Plate XXI, some of which are presented in Table 69, a high degree of accuracy is claimed. These experimental data cover the limits given both on Plate XXI and in Table 69. The

writer is confident that they will give accurate results under conditions "which are met in the operation of a canal system." Mr. Lyman.

Little Additional Cost Brings Better and More Accurate Results.—The small additional cost of "constructing a channel of approach", instead of making "the enlargement of the up-stream portion of the stream bed to a sufficient area" (and Mr. Winsor might well have added to a sufficient depth), will, in the opinion of the writer, be much more than compensated by the additional convenience, accuracy, and satisfaction that will result. Mr. Winsor certainly appreciates the value of maintaining a large velocity of approach in this mountainous country, as does every other engineer who has had experience where streams carry great quantities of silt.

Information Concerning One Particular Weir More Needed Now Than Information Concerning a General Solution.—It is not clear why Mr. King thinks that, in order to be of general application, the solution to the weir problem "must provide for a channel of approach of any cross-section." Of course, conditions will arise which make it desirable to know what quantity of water with a given head is passing over some irregular crest having uncertain contractions. Great care has been taken and much space used to provide, in this paper, a means of measuring the quantity of water flowing over irregular weirs, and the results reached are satisfactory; but, when water is to be measured with a high degree of accuracy, the device to be used must be built for this purpose.

As the suppressed weir can be used under practically all conditions where accurate results are demanded, the writer thinks it better for hydraulicians, who are in a position to do so, to gather more information concerning the discharge over this weir than to attempt to get a "satisfactory general method of solution for all weir problems." Although this general information would be of interest and value, the writer regards the more scientific data as having a greater value.

The Greater the Velocity of Approach the Better.—As water rarely if ever flows over a weir without some velocity of approach, Mr. King's suggestion to secure data for this condition does not appeal to the writer as of great importance. In fact, as the greater the velocity of approach the more ideal is the weir for use where silt is carried in the water, would it not be better to gather information concerning weirs with smaller heights and therefore with still greater velocities of approach?

The Best Standard Weir for All Cases.—Taking into account the quantity and reliability of the data available, the range of heights covered by the series of weir experiments used herein (see list on Plate XXII), and the comparatively wide range of heads covered by these experiments, the writer must differ with Mr. Horton, and say that there is a weir "which is the best standard for all cases", and that it

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should be so considered by all engineers until data are furnished that will show some other weir at least equal, in its general application, to that without end contractions.

Simplicity of Method.—Mr. Horton's discussion argues in favor of making "the computation as simple as possible by using the $\frac{3}{2}$ power of the head, instead of causing the head to vary according to some other fractional or decimal power, which would require the use of logarithms, logarithmic paper, or a logarithmic slide-rule for practical computations," and he closes remarking "regretfully, as regards hydraulic science, 'Of the making of many formulas there is no end'".

Although this paper presents a great array of formulas, these are used, and are intended only to be used, for the purpose of preparing diagrams similar to those shown on Plate XXI and in Table 69, from which, without calculation, discharges are obtained.

Mr. Smith states that he cannot agree that the diagrams herein presented are "in the interests of simplicity." Then he adds "errors in taking results from the diagrams can easily exceed the discrepancy between the author's and the Francis formulas." The writer believes that Mr. Smith will change his view after examining Table 72. If he takes the results obtained from Plate XXI and checks them by Table 69, he will probably be surprised at the smallness of the "discrepancy."

Discharge Proportional to Length of Crest.—Mr. Smith says, again, "one assumption made at the outset of the paper is perhaps open to question. It is that the discharge is proportional to the length of crest."

In reply, attention is drawn to the fact that the famous hydraulician, Bazin, made very careful experiments on his two standard weirs, one having a length of 1 m., the other having a length of 2 m. Concerning the results obtained, he writes:

"All needful precautions were taken to insure a flow under exactly the same conditions as over the 2-m. weir. The height of the weir remained the same. It might, however, be asked whether, in view of a somewhat different distribution of the velocities, or of an increase in the effect of friction against the side-walls, the results would be strictly comparable. We have failed to observe any appreciable difference between the two series."

In the new canals of the Hydraulic Laboratory of the University of Utah, the writer hopes to be able to determine whether or not the discharge is proportional to the length of crest, and also if the friction on the side-walls, where the velocity is comparatively small, is a factor that must be taken into account when a high degree of accuracy is desired.

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Mr. Smith asserts, with others, that "the Cippoletti weir discharge is proportional to the length." If this were true in the sense that the same quantity of water passes over each and every unit of length of crest of this weir, it could be used for a dividing device, as can the weir without end contractions. A moment's reflection shows that water flows through the triangular areas outside of the ends of the crest proper; it follows, therefore, that the discharge is not proportional to the length. For this reason the Cippoletti weir cannot be used as an accurate dividing device.

Simplicity of Table 69.—The table of discharges (Table 69) is believed to be quite as simple as the table referred to by Mr. Smith, which he says "is so simple that an engineer is not required to interpret it."

Height of Crest of Cippoletti Weir Omitted.—If the "height" of the crest of the Cippoletti weir above the bottom of the channel of approach is not specified, then the statement made by Mr. Smith that the error, under the three conditions he names, "will be less than 1%" is far from correct.

One Type of Weir Should Be Established by Legislation.—"Exception," Mr. Smith says, "should be taken" to the "proposition to establish one type of weir for universal use by legislation." Every engineer who has been connected with power-plant work, involving, as it does and must, the efficiency of hydraulic motors, has learned that, in order to determine in a satisfactory way the quantity of water used, it is wise, if not positively necessary, to specify how the water is to be measured. When, as is the case these days, "bonuses and forfeits" involving thousands of dollars are based on variations of discharge as small as one-tenth of 1%, as stated by Mr. Williams, it is certainly advisable, by legislation or otherwise, to fix some one standard form of weir measurement. In such a case it is as important, perhaps, to get uniform results as to get results which are accurate.

Mr. Smith says, "The best method for a particular case depends on the surrounding conditions." The writer holds that the best method for all cases is that by which the same quantity of flowing water can be measured again and again by different trained observers, each measurement giving the same result. This is especially true if the result is an accurate one.

That the Utah law establishing uniform cross-sections for roads is an unwise one is a fact that has no bearing on this discussion.

Results Inaccurate When not Based on Experiment.—Can Mr. Smith know, or can he or any one else find out, at any reasonable cost, what degree of accuracy he obtained when measuring water with the special weir described in his discussion? The channel of approach, he says, was "scoured out to a depth of from 1 to 2 ft." Thus great additional uncertainty was introduced into the results.

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Modified Venturi Meter.—The writer agrees with Mr. Smith that "in the western part of the United States it is likely that a modification of the Venturi water meter will come into use to a considerable extent." Experimental data on a device of this sort will be welcomed.

Undershot Weir.—Another device, which may be used with great satisfaction in the near future, is the "inverted weir" without end contractions, that is, a notch in the bottom of a diaphragm across the channel, with contraction at the top only. Through such a notch or under such a "weir", sediment would pass readily. In order to make this structure a very valuable measuring device, it only remains for some one to determine the relation between the quantity of water flowing through the notch and the difference in elevation between the surface of the water above and the surface of the water below the diaphragm.

The Cippoletti or the Suppressed Weir?—Mr. Jarvis writes:

"If a single type of weir were adopted, either the Cippoletti or the sharp-crested rectangular weir without end contractions would doubtless serve best in irrigation practice."

For accurate work, where measurements are to be taken over comparatively long periods and under varying conditions, practically every good reason is in favor of the suppressed and against the Cippoletti weir.

Weir for Quick Rough Work.—A case cited by Mr. Jarvis is a good one in favor of the Cippoletti weir; that is, where a portable device of this sort is used by a ditch rider. When such rough and hurried measurements give the accuracy desired, it may be wise to use the Cippoletti weir.

Measuring Flume and Current Meter.—Mr. Jarvis draws attention to the use of the measuring flume and the current meter. In canals where the slope is slight, these can be wisely used.

Maximum Discharge, Minimum Obstruction.—The writer is grateful to Mr. Jarvis for the clear statement that "the rectangular weir without end contractions provides the maximum width of crest, and the minimum obstruction to the flow in the canal."

Difficulties Encountered in Accurate Measurement of Heads.—The value of this paper is greatly increased by the keen comments of Mr. Williams. He points out errors which are introduced into weir work through inaccuracies in measuring heads. He also presents much original matter relating to differences which occur when the same head is measured on the same weir at the same time by different persons and in different ways. He points out specifically the difficulties encountered when an attempt is made to measure heads carefully.

Conclusion.—In conclusion the writer could hardly do better, perhaps, than refer to the six reasons given near the beginning of this

paper setting forth the advantages in practice of using weirs without end contractions. In the language of Mr. Jarvis, these weirs "provide the maximum width of crest and the minimum obstruction to the flow in the canal." This is a vital matter where streams carry great quantities of silt. Mr. Lyman.

New practical features added since the paper was written are the use of flush-gates in the bulkhead which carries the permanently fixed crest (Figs. 50 and 51); suggestive definite dimensions for weirs to be used for measuring certain quantities of water, and the quantities of concrete required to build the structures recommended (Table 71); and a table giving discharges over all sharp-crested weirs without end contractions within the limits of experimental data (Table 69).

The writer is more strongly convinced than ever that when irrigation engineers and practical hydraulicians have given careful consideration to the matters contained herein they will agree with him that, for general application, where accurate practical information is wanted, and where any weir can be used for measuring water, that without end contractions will answer the purpose best.

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Paper No. 1305

STEEL STRESSES IN FLAT SLABS.

By H. T. Eddy, Esq.*

With Discussion by Messrs. Edward Godfrey, H. E. Eckles, Sanford E. Thompson, L. J. Mensch, W. K. Hatt, George S. Binckley, C. A. P. Turner, and H. T. Eddy.

The question of the percentage of reinforcement in flat slabs is important economically as well as theoretically. It is, moreover, a question respecting which there seems to be some misconception as to the principles which should be applied. Shall the concrete and steel be balanced as perfectly as possible; or, if not, which shall be made the stronger? The uninformed have usually insisted on over-reinforcement, so that the concrete would be the weaker. If it is possible to determine the actual stresses in the steel with precision, there is no doubt that slabs should be designed so that the steel will give way first. Concrete is an unreliable and fragile material compared with steel. Under-reinforcement, however, will make security depend on the steel, which has known properties capable of precise computation. When the steel begins to yield, that action will cause the concrete to fracture to some extent, but the structure will exhibit toughness, and will not be subject to sudden collapse. In case of over-reinforcement, where the concrete is crushed and gives way first, the structure is in danger of much more sudden and unforeseen failure.

That these conclusions coincide with those of economic design appears from the following consideration of the stresses in steel and

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concrete, as given by Turneaure and Maurer's equations for the equal resisting moments of steel and concrete per unit of width of slab regarded as a beam, namely,

$$M_s = f_s\, Ajd = f_s\, pjd^2, \text{ and } M_c = \frac{1}{2} f_c\, kjd^2,$$

in which $A = pd$ is the cross-section of the reinforcement per unit of width of slab, $kd =$ the depth of the neutral axis, and $jd =$ the depth of the steel below the center of compression of the concrete. As these moments are equal, the ratio of the stresses in steel and concrete is,

$$\frac{f_s}{f_c} = \frac{k}{2\,p},$$

in which p is the steel ratio of reinforcement. Using Turneaure and Maurer's tabular values of k, in case $n = \frac{E_s}{E_c} = 15$, and assuming that $f_c = 500$ lb. per sq. in., and $f_s = 16\,000$ lb. per sq. in., the values of stresses in steel and concrete, respectively, shown in Table 1, are found.

TABLE 1.

$n = 15$.		$\frac{f_s}{f_c} = \frac{k}{2\,p}$.	For $f_c = 500$ $f_s = \frac{250\,k}{p}$.	For $f_s = 16\,000$ $f_c = \frac{32\,000\,p}{k}$.
p.	k.			
0.0025	0.25	50	25 000	320
0.005	0.32	32	16 000	500
0.010	0.42	21	10 500	762
0.015	0.48	16	8 000	1 000
0.020	0.525	13	6 550	1 220

Table 1 shows how small the percentage of reinforcement should be (less than one-half of 1%) in order that, in case of under-reinforcement, the steel should be used with reasonable economy; also, in case of over-reinforcement, how great the stresses in the concrete become in order to develop the strength of the steel. Every consideration of correct design, safety, and economy seems to demand a very low percentage of steel, verging on under-reinforcement, rather than on any attempt at over-reinforcement.

If these principles are adopted as controlling the design of flat slabs, the all-important and crucial question is that of unit stresses in the steel. The writer hopes that the following discussions of tests

for actual stresses in several large buildings, as compared with computed theoretical stresses, may assist in supplying a satisfactory basis for computation and design such as has not been available hitherto.

The Northwestern Glass Company Building.

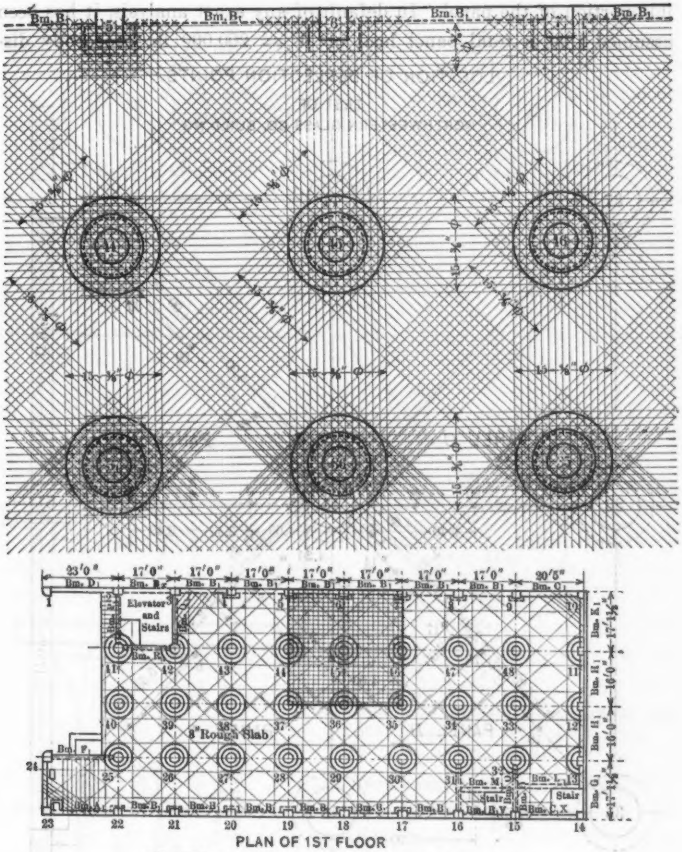
The mushroom, flat slab floor in the Northwestern Glass Company Building, in Minneapolis, Minn., was tested by Mr. F. R. McMillan, on May 12th to 24th, 1913. The pressure for immediate occupancy of the building by the owners was such as to compel the completion of the test at an undesirably early date after the slab, which had been frozen all winter, had sweated out, and had had an opportunity to commence to become cured. It is estimated that the condition of the slab at the beginning of the test was perhaps such as would be expected at the end of a period of 45 days favorable for curing.

Appendix A contains the particulars of the test conducted by Mr. McMillan, whose results are used in this discussion. The equations applied in the computations may be found in the writer's book, "The Theory of Flat Slabs."*

The maximum load on Panel D amounted approximately to 185 000 lb. This was placed on the four quarters of the panel, with open passageways, 1.5 ft. wide, across the middle of the panel each way, and with open spaces around the columns at each corner arranged so that only 181.5 sq. ft. of the total panel area of 16 by 17 ft. = 272 sq. ft., were actually covered by the loading. With such an arrangement of the loading it was difficult to determine just what should be assumed as the equivalent uniform load. The difficulty is increased by the fact that, no matter what load is placed on the mushroom heads, it will have little effect in increasing either stresses or deflections. Consequently, attention must be given principally to the equivalence of the loading not located over the heads.

The open passageways decreased the effectiveness of the actual loading, and the open spaces over the heads increased its effectiveness in a very complicated manner when one also considers their stiffness relative to the remainder of the panel area. It will be evident, however, that, on the whole, the loading as actually applied, was more effective in producing deflections and stresses than if the total actual

* The numbers of the equations in this paper are the same as those in "The Theory of Flat Slabs."



PLAN OF 1ST FLOOR

REINFORCING PLAN OF TEST PANELS

TEST OF

NORTHWESTERN GLASS CO. BUILDING.

FIG. 1.*

* **All the Figures in this paper relating to this test are taken from the report by Mr. McMillan, other parts of which are given in Appendix A.**

load had been distributed uniformly over the panel. This takes into account the fact that the load, almost exclusively, was on the more flexible portion of the panel. In default of any exact analysis, it has been assumed that a total panel load of $W = 200\,000$ lb. uniformly dis-

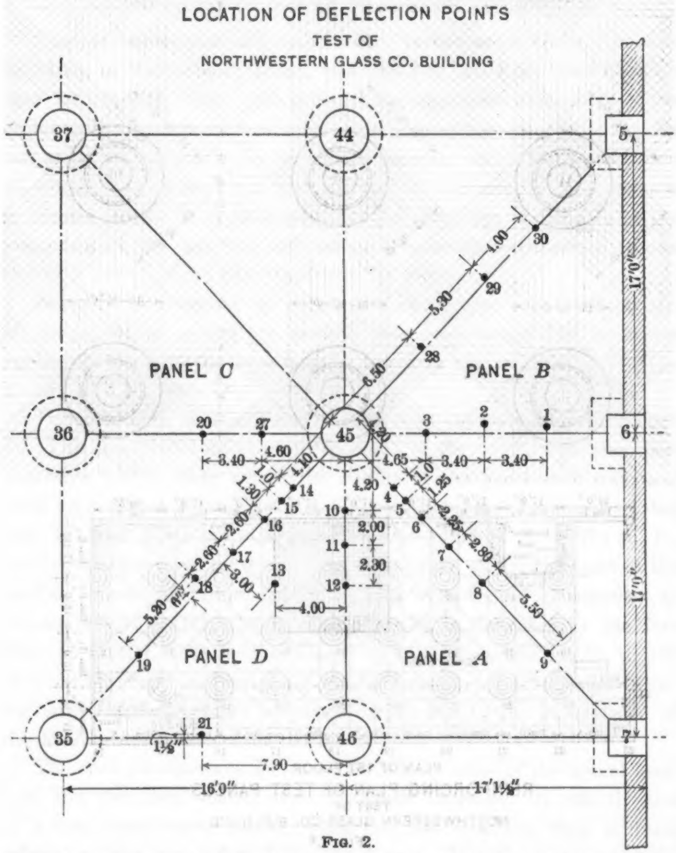
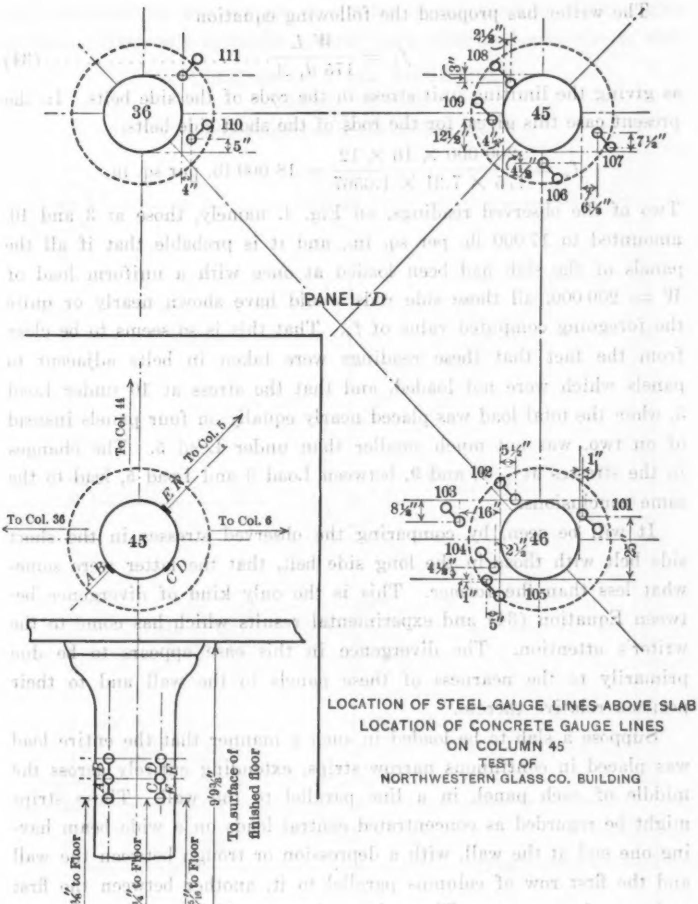


Fig. 2.

tributed would cause deflections and stresses at least as great as those due to the actual load of 185 000 lb.

This value of W will be used in the computations as equivalent to the actual load, although it is evident that, in computing deflections,



111
36
110
5"
4"
2⅛"
108
109
45
12⅛"
4⅛"
7½"
107
106
6⅛"
PANEL D
To Col. 11
To Col. 5
E F
To Col. 36
To Col. 6
45
A B
C D
5½"
1"
102
103
8⅛"
16"
101
46
104
4⅛"
1"
105
5"
6'3⅞" to Floor
6'0¼" to Floor
6'1⅝" to Floor
9'9¾" To surface of finished floor
LOCATION OF STEEL GAUGE LINES ABOVE SLAB
LOCATION OF CONCRETE GAUGE LINES ON COLUMN 45
TEST OF NORTHWESTERN GLASS CO. BUILDING

FIG. 3.

a value of W, somewhat different from that needed in computing stresses, might be required.

The writer has proposed the following equation

$$f_s = \frac{W L}{175\, d_1\, A} \qquad \text{(34)}$$

as giving the limiting unit stress in the rods of the side belts. In the present case this gives, for the rods of the short side belts,

$$f_s = \frac{200\,000 \times 16 \times 12}{175 \times 7.31 \times 1.6567} = 18\,000 \text{ lb. per sq. in.}$$

Two of the observed readings, on Fig. 4, namely, those at 3 and 10, amounted to 17 000 lb. per sq. in., and it is probable that if all the panels of the slab had been loaded at once with a uniform load of $W = 200\,000$, all these side rods would have shown nearly or quite the foregoing computed value of f_s. That this is so seems to be clear from the fact that these readings were taken in belts adjacent to panels which were not loaded, and that the stress at 10 under Load 3, when the total load was placed nearly equally on four panels instead of on two, was not much smaller than under Load 5. The changes in the stresses at 7, 8, and 9, between Load 3 and Load 5, lead to the same conclusions.

It will be seen, by comparing the observed stresses in the short side belt with those in the long side belt, that the latter were somewhat less than the former. This is the only kind of divergence between Equation (34) and experimental results which has come to the writer's attention. The divergence in this case appears to be due primarily to the nearness of these panels to the wall and to their position relative thereto.

Suppose a slab to be loaded in such a manner that the entire load was placed in continuous narrow strips, extending entirely across the middle of each panel, in a line parallel to the wall. These strips might be regarded as concentrated central loads on a wide beam having one end at the wall, with a depression or trough between the wall and the first row of columns parallel to it, another between the first and second rows, etc. These depressions would be much more pronounced than any crossing them perpendicularly to the wall.

The wall and Load 5 evidently had an effect of this kind, which caused the stresses in the short side belts perpendicular to the wall to exceed those in the long side belts.

Indeed, slab action in general may be described partly as the attempted mechanical superposition of one set of parallel depressions and elevations on another set of similar corrugations at right angles to them. Such sets mutually support each other and give rise to slab

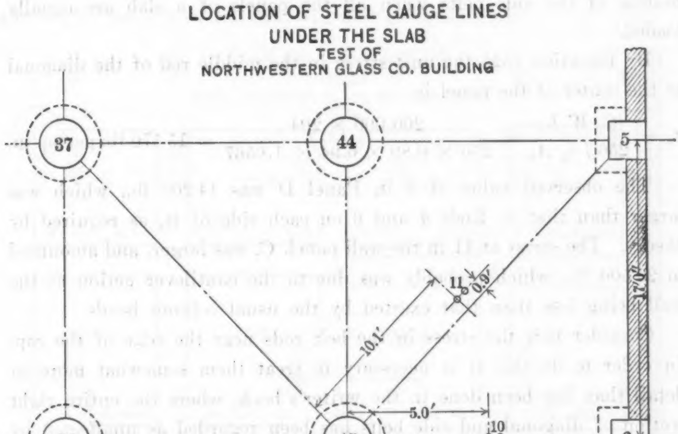


FIG. 4.

action, but any action which interferes with and disturbs the regularity of this superposition needs consideration. A wall support is such an interference, for it completely destroys the regular sequence of depressions and elevations at the end of one set and thus intensifies

those in the other set. The case just discussed is evidently of this character.

The writer has shown* that the stresses to be expected in the diagonals at the center of the panels are somewhat less than those in the middle of the side belts when all the panels of a slab are equally loaded.

By Equation (52) the unit stress in the middle rod of the diagonal at the center of the panel is

$$f_s = \frac{C_1 \, W \, L_1}{256 \, j \, d_2 \, A_1} = \frac{200\,000 \times 204}{256 \times 0.89 \times 6.94 \times 1.6567} = 15\,570 \text{ lb. per sq. in.}$$

The observed value at 5 in Panel D was 14 200 lb., which was larger than that in Rods 4 and 6 on each side of it, as required by theory. The stress at 11 in the wall panel, C, was larger, and amounted to 20 500 lb., which probably was due to the cantilever action at the wall being less than that exerted by the usual column heads.

Consider now the stress in the belt rods near the edge of the cap. In order to do this it is necessary to treat them somewhat more in detail than has been done in the writer's book, where the entire right section of diagonal and side belts has been regarded as unaffected by the mass of the cap, which forms a large boss, integral with the slab, and, as far as it extends, renders the belt much less extensible and compressible than it would be without it. The edges of each belt are so far from the edge of the cap as to permit of the assumption that the rods at the edge have the same stresses as they would have were the cap not present; but those rods which are tangent to the cap will have their elongations and stresses reduced thereby, and the rods beside the cap and intermediate between these and the edges of the belts will have their stresses reduced in proportion to their proximity to the cap.

The case is different, however, with rods which cross the edge of the cap nearly perpendicularly. Although that part of the length of the rod which is inside the cap has its elongation prevented by the mass of the cap, the part outside must have its elongation correspondingly increased to compensate for this loss, and, on the whole, be equal to that of the rods beside it. The stresses in the middle rods of each belt, consequently, are increased abnormally for this reason just as they leave the cap. Instead of attempting to determine this increase by

* "The Theory of Flat Slabs."

some intricate investigation, it will simply be assumed that the stress at this point in the middle rod does not exceed that in the outside rod of the belt at a point opposite the center of the cap. This must be very nearly the fact, and the stress in the outside rod at this point, as affected by the size of the cap, will now be considered.

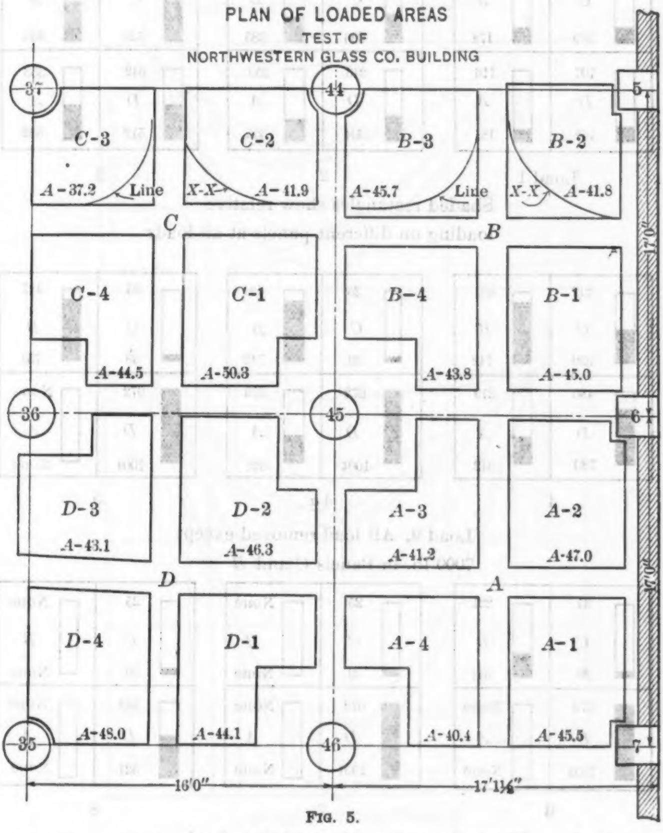


FIG. 5.

As the cap is integral with the slab and may be taken as horizontal at the edge of the cap instead of at the center of the column, as has been assumed in the writer's equations, the position of the points of inflection or contraflexure of the rods will be nearer the center of the

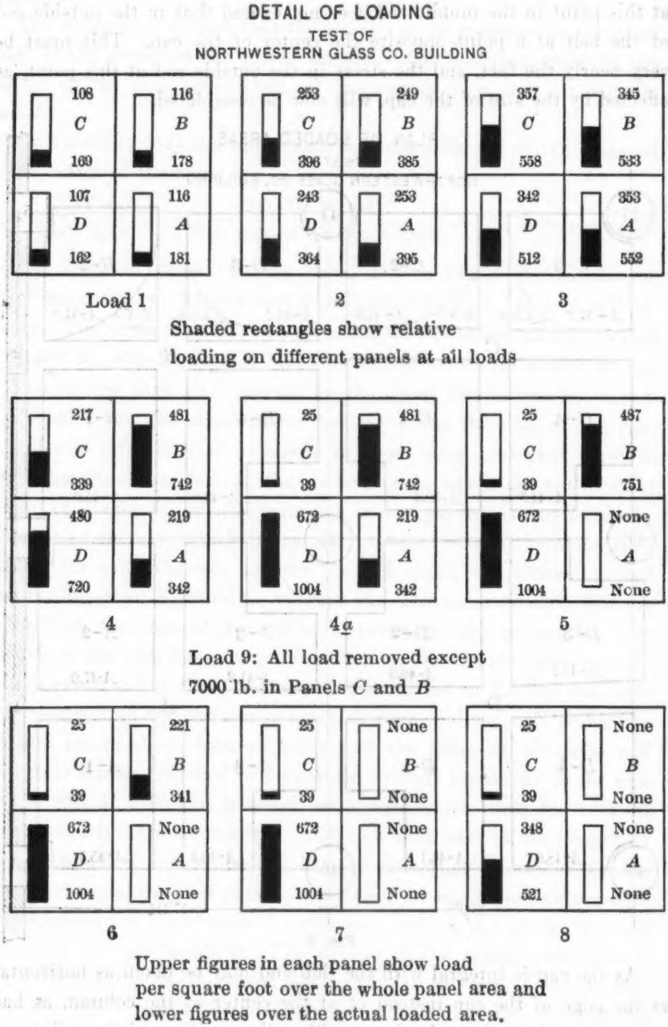

DETAIL OF LOADING
TEST OF
NORTHWESTERN GLASS CO. BUILDING
108 C 169
116 B 178
107 D 162
116 A 181
Load 1
253 C 396
249 B 385
243 D 364
253 A 395
2
357 C 558
345 B 533
342 D 512
353 A 552
3
Shaded rectangles show relative
loading on different panels at all loads
217 C 339
481 B 742
480 D 720
219 A 342
4
25 C 39
481 B 742
672 D 1004
219 A 342
4a
25 C 39
487 B 751
672 D 1004
None A None
5
Load 9: All load removed except
7000 lb. in Panels C and B
25 C 39
221 B 341
672 D 1004
None A None
6
25 C 39
None B None
672 D 1004
None A None
7
25 C 39
None B None
348 D 521
None A None
8
Upper figures in each panel show load
per square foot over the whole panel area and
lower figures over the actual loaded area.

FIG. 6.

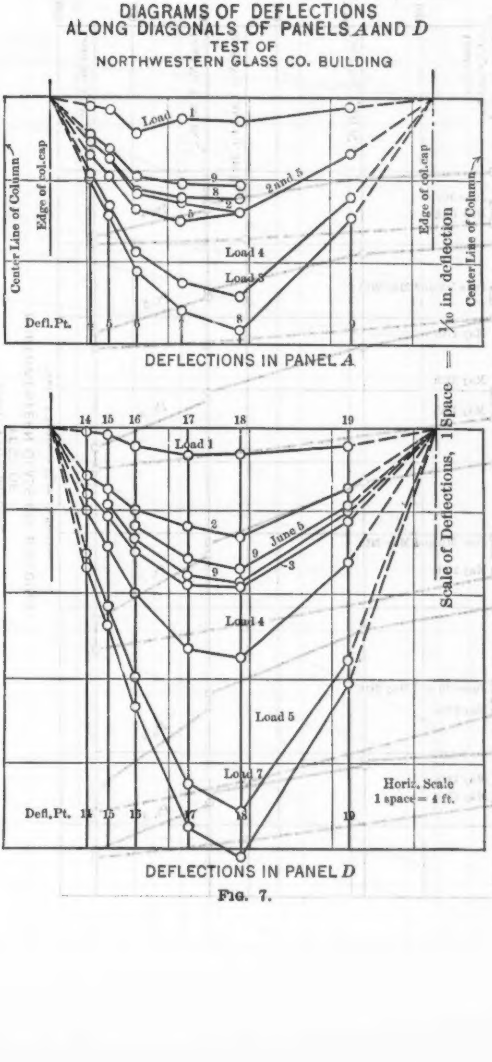

DIAGRAMS OF DEFLECTIONS
ALONG DIAGONALS OF PANELS A AND D
TEST OF
NORTHWESTERN GLASS CO. BUILDING
Load 1
2 and 5
Load 4
Load 3
Center Line of Column
Edge of col. cap
Defl. Pt.
DEFLECTIONS IN PANEL A
Scale of Deflections, 1 Space = 1/10 in. deflection
14 15 16 17 18 19
June 5
Load 5
Load 7
Horiz. Scale
1 space = 4 ft.
DEFLECTIONS IN PANEL D

FIG. 7.

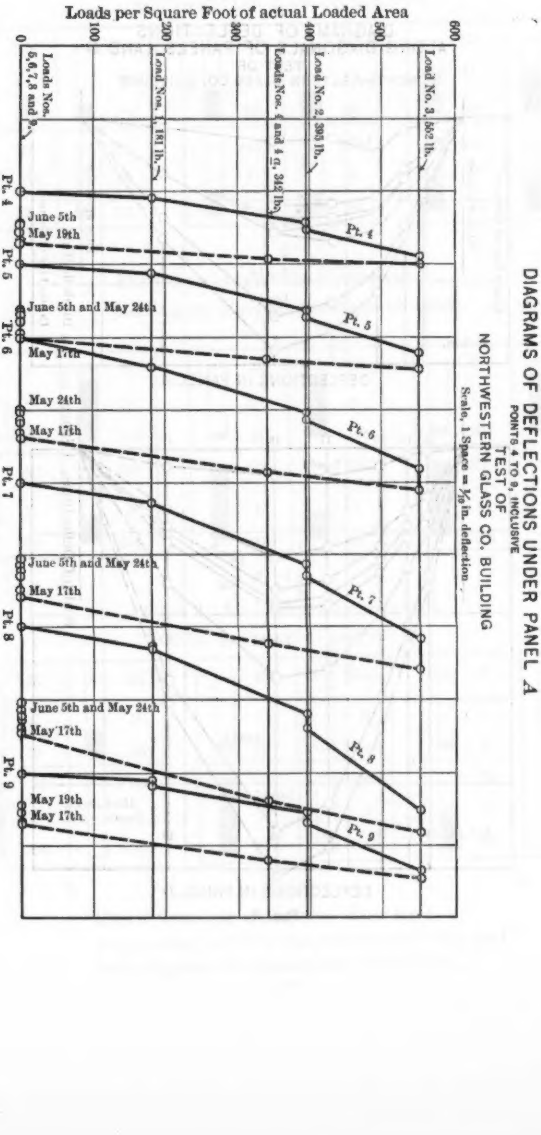

DIAGRAMS OF DEFLECTIONS UNDER PANEL A
POINTS 4 TO 9, INCLUSIVE
TEST OF
NORTHWESTERN GLASS CO. BUILDING
Scale, 1 Space = 1/10 in. deflection
Loads per Square Foot of actual Loaded Area
0
100
200
300
400
500
600
Loads Nos. 5, 6, 7, 8 and 9.
Load Nos. 1, 181 lb.
Loads Nos. 4 and 4 a, 342 lb.
Load No. 2, 385 lb.
Load No. 3, 552 lb.
Pt. 4
June 5th
May 19th
Pt. 5
June 5th and May 24th
Pt. 6
May 17th
May 24th
May 17th
Pt. 7
June 5th and May 24th
May 17th
Pt. 8
June 5th and May 24th
May 17th
Pt. 9
May 19th
May 17th.

FIG. 8.

DIAGRAM OF DEFLECTIONS UNDER PANEL *B*,
POINTS 1, 2, 3, 28, 29, AND 30,
TEST OF
NORTHWESTERN GLASS CO. BUILDING
Scale, 1 Space = $\frac{1}{16}$ in deflection

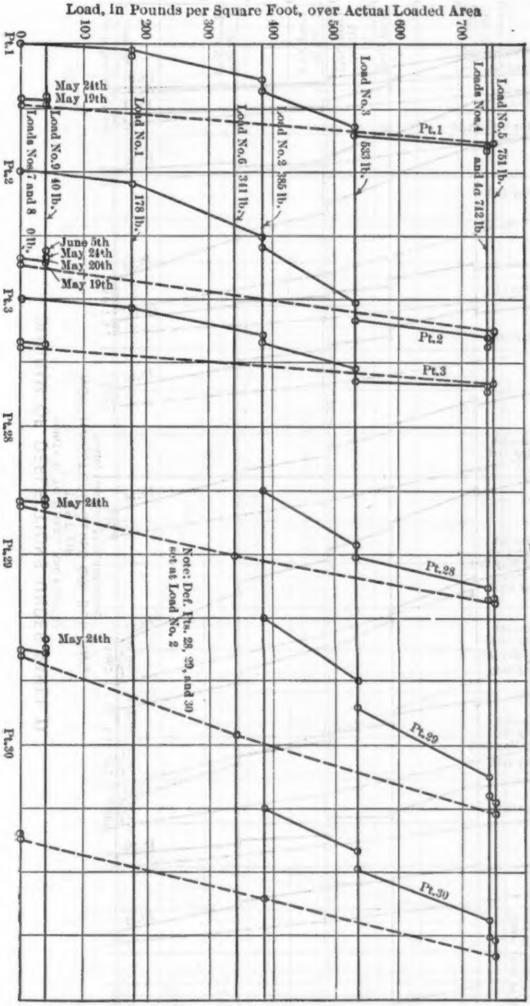


FIG. 9.

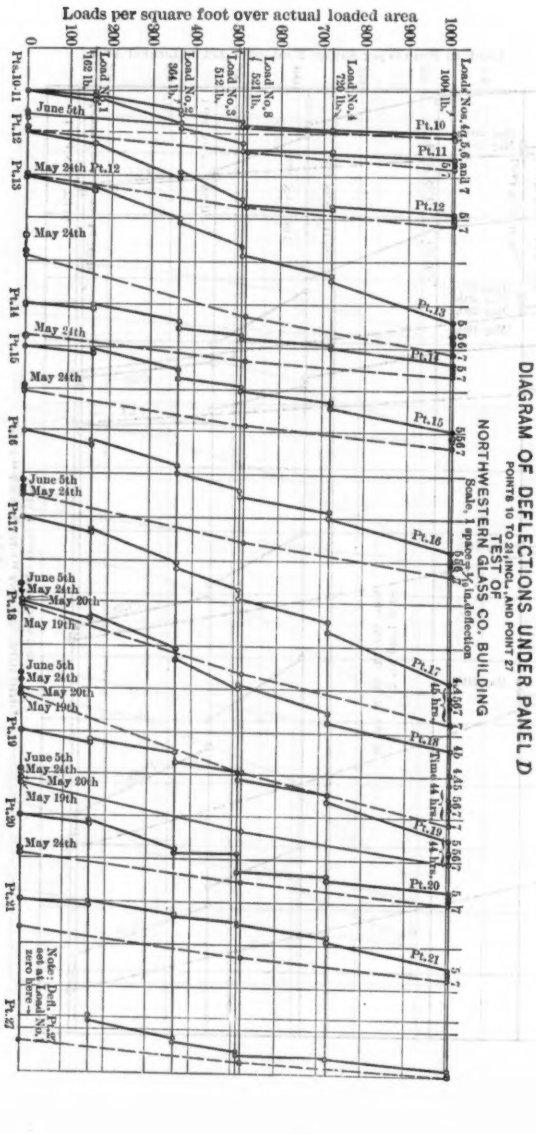


DIAGRAM OF DEFLECTIONS UNDER PANEL D
POINTS 10 TO 21, INCL., AND POINT 27
TEST OF
NORTHWESTERN GLASS CO. BUILDING
Scale, 1 space = 1/16 in. deflection
Loads per square foot over actual loaded area
0
100
200
300
400
500
600
700
800
900
1000
Load No. 1
162 lb.
Load No. 2
364 lb.
Load No. 3
512 lb.
Load No. 8
521 lb.
Load No. 4
720 lb.
Loads Nos. 4a, 5, 6, and 7
1004 lb.
Pts. 10-11
Pt. 12
Pt. 13
Pt. 14
Pt. 15
Pt. 16
Pt. 17
Pt. 18
Pt. 19
Pt. 20
Pt. 21
Pt. 27
Pt. 10
Pt. 11
Pt. 12
Pt. 13
Pt. 14
Pt. 15
Pt. 16
Pt. 17
Pt. 18
Pt. 19
Pt. 20
Pt. 21
June 5th
May 24th
May 20th
May 19th
Time 44 hrs.
Note: Defl. Pt. 27 set at Load No. 1 zero here

FIG. 10.

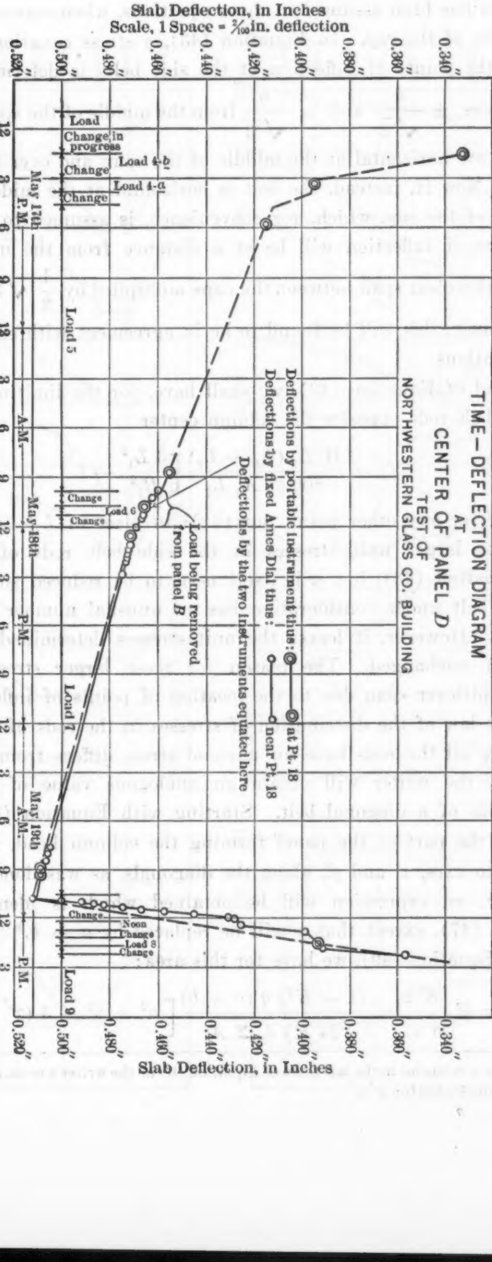

TIME—DEFLECTION DIAGRAM
AT
CENTER OF PANEL D
TEST OF
NORTHWESTERN GLASS CO. BUILDING
Deflections by portable instrument thus: at Pt. 18
Deflections by fixed Ames Dial thus: near Pt. 18
Deflections by the two instruments equated here
Load being removed from panel B
Slab Deflection, in Inches
Scale, 1 Space = 2/100 in. deflection
0.340″
0.360″
0.380″
0.400″
0.420″
0.440″
0.460″
0.480″
0.500″
0.520″
Slab Deflection, in Inches
Load
Change in progress
Change
Load 4-b
Load 4-a
Change
Load 5
Change
Load 6
Change
Load 7
Load 8
Change
Noon
Change
Load 8
Change
Load 9
12
3
6
9
12
3
6
9
12
3
6
9
12
3
6
9
12
3
May 17th
P.M.
A.M.
May 18th
P.M.
A.M.
May 19th
P.M.

FIG. 11.

span than has been assumed in those equations, when account is taken of the size of the cap. In Equation (43), a stress equation, the position of the points of inflection of the side belts is determined to be at distances $\pm \frac{a}{\sqrt{3}}$ and $\pm \frac{b}{\sqrt{3}}$ from the middle of the span, because the belts are horizontal at the middle of the span and over the column centers. Now if, instead, the belt is horizontal at the middle, and at the edge of the cap, which, for convenience, is assumed to be square, the points of inflection will be at a distance from the mid-span of one-half the clear span between the caps multiplied by $\frac{1}{3} \sqrt{3} = 0.577$. Subsequently, this will be found to be in agreement with experimental determinations.

Instead of Equation (43), we shall have, for the limiting stress in the side belt rods opposite the column center

$$f_s = \frac{W L_1 (L_1 + L_2)}{800\, d_3 A_1 L_2} \left[\frac{3 L_1^2}{B_1^2} - 1\right] \ldots\ldots\ldots\ldots (a)$$

To find f_s at any other point, put $(2x)^2$ in place of L_1^2. This equation gives larger unit stresses in the side belt rods of the head than Equation (43), but will itself need to be reduced somewhat in case the belt under consideration has an unusual number of laps in the head. However, it leaves the unit stresses determined by Equation (34) unchanged. The reason for these larger stresses is the longer cantilever span due to the location of points of inflection.

As the law of the distribution of stresses in the rods of a diagonal belt where all the rods have an unequal stress differs from that in a side belt, the writer will obtain an analogous value of the stress in the rods of a diagonal belt. Starting with Equation (39), which refers to the part of the panel forming the column head, and transforming to axes, x' and y', along the diagonals, as was done in Equation (47), an expression will be obtained which is identical with Equation (47), except that g will be replaced by $a + b$.* Hence, in place of Equation (49), we have for this area:

$$\pm e' = id \frac{\delta^2 z}{\delta x'^2} = \frac{(1 - K^2)\, q\, (a + b)}{24\, E j\, d_3\, \Sigma A} \left[a^2 + b^2 - 3\,(x'^2 + y'^2)\right]$$

* There is a misprint in the last term of Equation (47) in the writer's book, and $x'^2 + y'^2$, should be substituted for $x'^2 y'^2$.

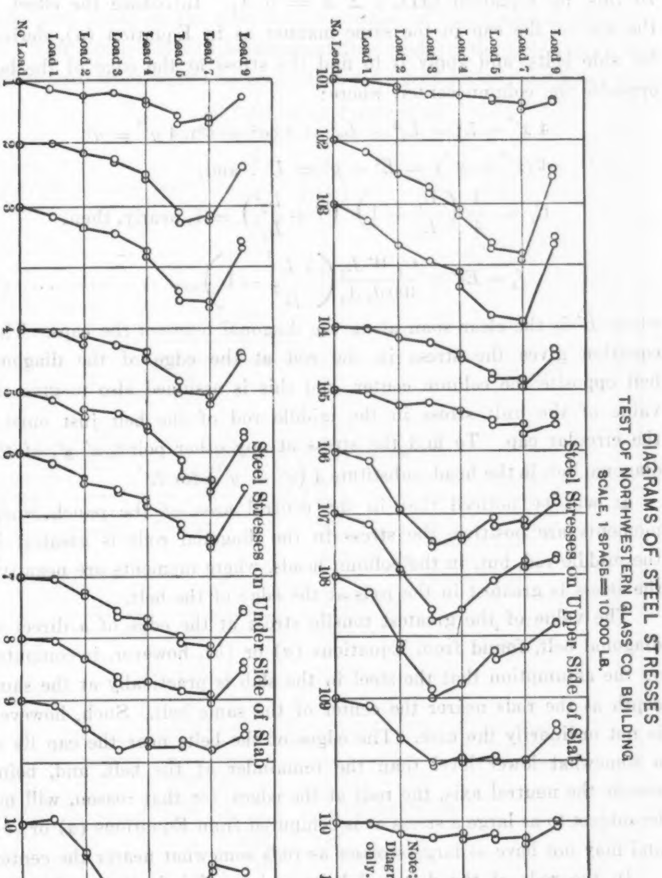

DIAGRAMS OF STEEL STRESSES
TEST OF NORTHWESTERN GLASS CO. BULDING
SCALE, 1 SPACE = 10,000 LB.
Steel Stresses on Upper Side of Slab
Note: Diagrammatic representation only. Ordinates not to scale
No Load
Load 1
Load 2
Load 3
Load 4
Load 5
Load 7
Load 9
101
102
103
104
105
106
107
108
109
110
111
Steel Stresses on Under Side of Slab
1
2
3
4
5
6
7
8
9
10
11

Fig. 12.

In this, by Equation (41), $j \Sigma A = 6 A_1$. Introduce the effect of the size of the cap in the same manner as in Equation (*a*), derived for side belts, and apply it to find the stress at the edge of the belt opposite the column center, where:

$$4 x'^2 = L^2 = L_1^2 + L_2^2 = 4 (a^2 + b^2), 4 y'^2 = g^2$$

$$4 (x'^2 + y'^2) = L^2 + g^2 = L'^2, \text{ and,}$$

$$C_1 = \frac{1}{4} \left(\frac{L_1}{L_2} + 1\right) \left(1 + \frac{L_2^2}{L_1^2}\right) = 1, \text{ nearly, then,}$$

$$f_s = Ee' = \frac{C_1 W L_1}{400 d_3 A_1} \left(\frac{3 L'^2}{B'^2} - 1\right) \text{..................} (b)$$

where B' is the clear span along the diagonal between the caps. This equation gives the stress in the rod at the edge of the diagonal belt opposite the column center, and this is assumed also to give the value of the unit stress in the middle rod of the belt just outside the circular cap. To find the stress at any other point, x' y', of the diagonal belt in the head, substitute $4 \left(x'^2 + y'^2\right)$ for L'^2.

It will be noticed that in the central area of the panel, where moments are positive, the stress in the diagonal rods is greatest in the middle rod, but, in the column heads, where moments are negative, the stress is greatest in the rods at the edge of the belt.

The value of the greatest tensile stress at the edge of a direct or diagonal belt, found from Equations (*a*) or (*b*), however, is computed on the assumption that the steel in the slab is practically at the same depth as the rods nearer the center of the same belt. Such, however, is not ordinarily the case. The edges of the belts near the cap lie at a somewhat lower level than the remainder of the belt, and, being nearer the neutral axis, the rods at the edges, for that reason, will not be subject to as large a stress as is computed from Equations (*a*) or (*b*), and may not have as large stresses as rods somewhat nearer the center.

In the rods of the diagonal belts, points of inflection and points of equal stress lie on circles with centers at the center of the panel, so that the points of inflection of the side belts which lie on lines perpendicular to the sides do not coincide with the points of inflection of the diagonal rods.

In this test no observations were made on the rods of the side belts by which any comparison can be made with Equation (*a*), but several points on the diagonals near the edges of the caps were ob-

served, which may be estimated by Equation (*b*), though no observations were made on the rods near the edges of the diagonal belts.

Under Load 3 it will be assumed that each of the four panels was loaded with what would be equivalent to a uniformly distributed total

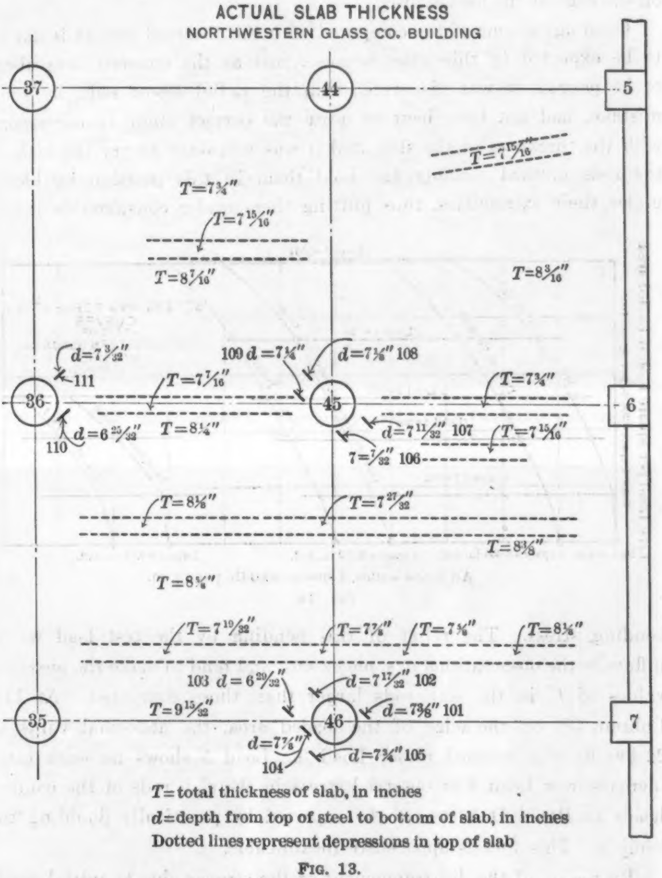


FIG. 13.

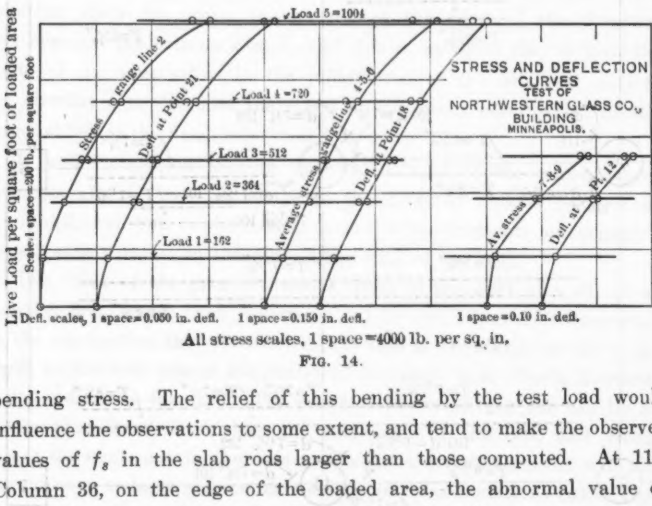
load of $W = 106\,250$ lb. By Equation (*b*) the limiting value of the unit stress in the outer rod is

$$f_s = \frac{106\,250 \times 204}{400 \times 6.5 \times 1.6567} \left(\frac{3 \times 86\,049}{60\,025} - 1\right) = 17\,000\text{* lb. per sq. in.}$$

* Subsequent recalculation in the discussion shows that $f_s = 19\,500$ lb., instead of 17 000 lb.

This is to be compared with the stresses observed on Fig. 3, most especially at 107 and 108, around the central cap of the loaded area, which were, respectively, 17 500 and 20 000 lb. Incidentally, comparisons should be made with observations at 102 and 111, at the caps on the edge of the loaded area.

Good agreement of the computed with the observed results is hardly to be expected in this case, because, just as the concrete was about to be poured, it was discovered that the radial elbow rods, by some mistake, had not been bent at quite the correct angle to correspond with the thickness of the slab, and it was necessary to pry the ends of the rods upward forcibly and hold them in this position by blocks under their extremities, thus putting them under considerable initial



All stress scales, 1 space=4000 lb. per sq. in.

FIG. 14.

bending stress. The relief of this bending by the test load would influence the observations to some extent, and tend to make the observed values of f_s in the slab rods larger than those computed. At 111, Column 36, on the edge of the loaded area, the abnormal value of 22 400 lb. was reached under Load 3. Load 5 shows no such large increase over Load 3 of observed stress in the slab rods of the column heads in Panel D as was to be expected by practically doubling the loading. This fact is apparently inexplicable.

By reason of the disarrangement of the stresses due to initial bending, as mentioned, it seems to be impossible to trace the stresses in the slab rods in greater detail in this test than has been done already, where a satisfactory concordant maximum stress in the slab rods has

FIG. 15.—SLAB REINFORCEMENT, NORTHWESTERN GLASS COMPANY BUILDING, MINNEAPOLIS.

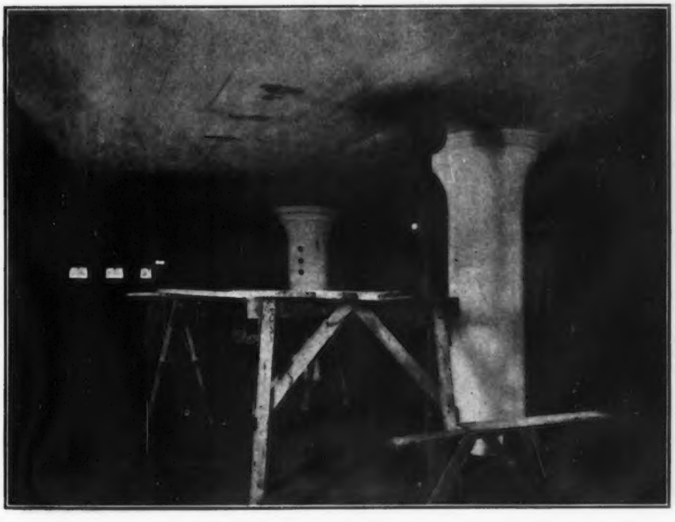


FIG. 16.—UNDER SIDE OF TEST PANELS, NORTHWESTERN GLASS COMPANY BUILDING, MINNEAPOLIS.

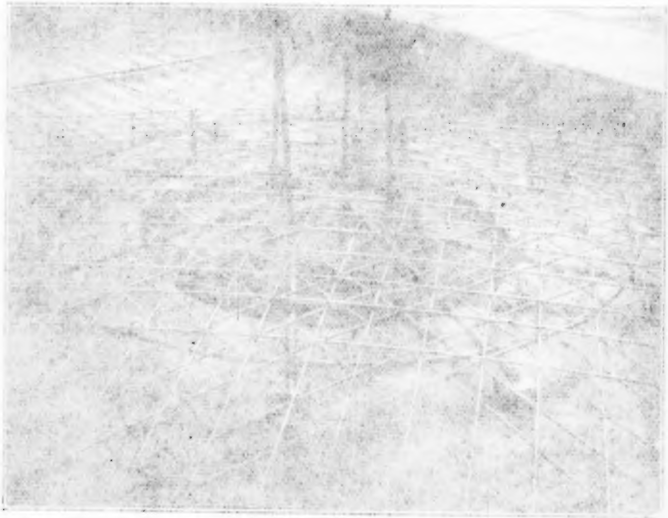
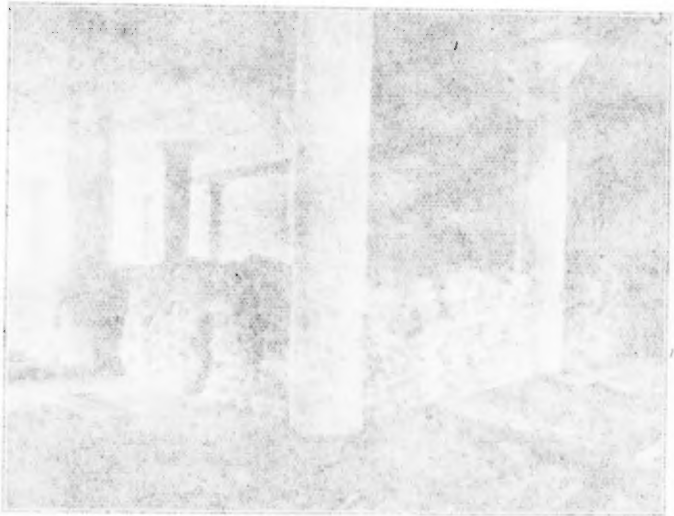


FIG. 12.—SLAB REINFORCEMENT, NORTHWESTERN GLASS COMPANY BUILDING, MINNEAPOLIS.

FIG. 13.—CLOSER VIEW OF TEST PANELS, NORTHWESTERN GLASS COMPANY BUILDING, MINNEAPOLIS.

FIG. 17.—UNIFORM LOAD OF 400 LB. PER SQ. FT. OVER FOUR PANELS. NORTHWESTERN GLASS COMPANY BUILDING.

FIG. 18.—TEST OF NORTHWESTERN GLASS COMPANY'S BUILDING BY MINNEAPOLIS BUILDING DEPARTMENT.



[illegible]

[illegible]

been reached. Consequently, a comparison of the computed with the observed deflections will now be considered. By Equation (71)

$$D_2 = \frac{W L_2 L_1^2}{4.4 \times 10^{10} A d_2^2}$$

in which $d_2 = 8 - 0.5 - 0.5625 = 6.9375$ in.

Hence $\quad D_2 = \dfrac{200\,000 \times 16 \times (17)^2 \times 1\,728}{4.4 \times 10^{10} \times 1.6567\,(6.9375)^2} = 0.4555$ in.

This agrees precisely with the value observed at 9 A. M. on May 18th under Point 18 at the center of Panel D. Practically the same deflection was found at the adjacent Point 17.

By Equations (61), (54), and (58), the deflection at the middle of the side belt is

$$D_1 = \frac{W L_1^3}{10^{10} A}\left[\frac{1}{10.7\ d_1^2} + \frac{\frac{L_1}{L_2} + 1}{60\ d_3^2}\right]$$

in which $d_1 = 7.31$ in. and $d_3 = 6.5$ in.

The values of the deflections are:

At Point 12: computed, 0.27 in., observed, 0.22 in.
" " 20 " 0.25 in., " 0.20 in.

The magnitude of the observed deflections in the side belts is less than those computed, for the same reason that the stresses are less, as before stated, although it is clear that the observed values would possibly reach those computed in case all the panels were loaded equally. This last remark is confirmed by considering the deflection under Load 3, which was nearly the same on all four panels, and equivalent to 106 250 lb., uniformly distributed, as found previously. The computed deflections would then be somewhat more than one-half of those already computed, and may be compared with the observed values, as follows:

At Point 12: computed, 0.15 in., observed, 0.16 in.
" " 20 " 0.13 in., " 0.13 in.

From which it may be concluded that the computed results would not be less than those observed if all the panels were equally loaded.

It seems clear that the continued increase of the deflection at Point 18 at the center of Panel D under Load 7 was influenced somewhat by the removal of the loading in Panel B, and that changes in the de-

flections of adjacent panels may be looked for along the prolongation of diagonals rather than across the sides of the panels. This is perhaps the first test of a mushroom slab in which it has been possible to discover interaction of panels. Possibly, however, this may be due to the initial stresses in the radial steel previously mentioned.

Fig. 7 is of interest as showing the dissymmetry in the right and left half spans of the diagonals in Panels A and D. In A the supporting action of the wall along the entire length of the side evidently prevented deflection to some extent in the half span nearest to it, so that Point 9 was deflected less than the corresponding point in the other half span. In Panel D Point 18 was deflected less than the corresponding Point 16, because all the slab rods are lapped over Column 35, and none over 45, as may be seen on Fig. 1.

Owing to the irregularities of loading and variations of stiffness due to the inclusion of the wall panels, as well as accidental variations of stiffness of construction due to initial bending stresses, and also the inequalities of laps, it is too much to expect formulas derived for entire uniformity of loading and equality of panels to apply more closely than these comparisons show.

The Deere and Webber Company Building.

The test of the Deere and Webber Building, in Minneapolis, Minn., was made on October 30th to November 5th, inclusive, 1910, by Mr. A. R. Lord.* The panel dimensions are 19 ft. 1 in. by 18 ft. 8 in., the slab is $9\frac{3}{16}$ in. thick, and the concrete was only 40 days old on October 30th. All the slab rods are $\frac{7}{16}$ in. round, with twelve rods in each side belt and fourteen in each diagonal. The head steel consists of the rectangular diamond frame with four rods extending entirely across it, which was at that time the typical form used by the Leonard Construction Company in such buildings. The column caps are 54 in. in diameter and the slab rods of all belts are 7.25 in. from center to center, so that the side belts may be taken to have a width of 80 in. and the diagonals nearly 95 in. The mean size of the cap is 0.24 *L*.

Various loads, uniformly distributed, were applied simultaneously to eight panels, which, with the omission of one corner panel, filled

*Reported by Mr. Lord in a paper printed in *Proceedings*, Nat. Assoc. of Cement Users, Philadelphia, Vol. VII, 1911.

out a square of nine panels. This gave opportunity to observe stresses in the slab rods in one panel at the center which was surrounded on all sides by equally loaded panels.

This discussion will refer mostly to the results observed when the greatest load was on all the eight panels at once. This load had a mean intensity of 350 lb. per sq. ft. The area of a panel is $356\frac{2}{9}$ sq. ft., and the total load $W = 350 \times 356\frac{2}{9} = 124\,678$ lb. The depth of the embedment of the various rods observed is given in the report, but, as the rods of a belt act together, it is not certain that the embedment of individual rods should be used in the formulas. It has been assumed that the fair mean values are

$$d_1 = 8.5 \text{ in.}, d_2 = 8 \text{ in., and } d_3 = 7.6 \text{ in.}$$

Applying Equation (34) to compute the unit stress in the slab rods on the short side of the panel at Point 14, of Fig. 3, in Mr. Lord's paper, which is the point where the greatest stress was found in any slab rod of a side belt between two loaded panels,

$$f_s = \frac{124\,678 \times 224}{175 \times 8.5 \times 12 \times 0.15} = 10\,000 \text{ lb. per sq. in.}$$

The observed stress at this point was 10 400 lb. The unit stress observed at Points 39, 40, and 14*a*,* on the short side belts, were somewhat less, but this is a satisfactory determination of the greatest stress at the middle of a short side belt.

Applying Equation (34) to compute the unit stress in the long side belt at Points 108 and 109,

$$f_s = \frac{124\,678 \times 229}{175 \times 8.5 \times 12 \times 0.15} = 10\,220 \text{ lb. per sq. in.}$$

The mean observed result was 6 600 lb. It is probable that this result, observed between two loaded panels bounded by panels not loaded, would be less than on the same belt in the two loaded panels beside it, on the principle that if only three successive spans of a continuous beam of many spans be loaded, the middle one will show less stresses and deflections than the end spans.

This, together with the fact that the inflection lines were much nearer to the center of the panel, was probably the reason that the observed result was so small. Apparently, however, there is no reason

* These refer to Fig. 3 of Mr. Lord's paper.

for the greatest unit stress being less on the long side of the panel than on the short side, when the belts are equal and the panels are so nearly square. In this particular the results of the test seem to be inexplicable unless the two bulkheads which extend nearly across the slab parallel to the long sides of the panels, one of which crosses the short side at 14, have this effect.

A similar discrepancy was found between the observed stresses on the long and short sides of the panel in other tests. It is certain that provision should be made on this long side for stresses at least as great as those on the short side.

Applying Equation (52) to the computation of the unit stress in the rods of the diagonal belt at the center of the panel at Points 3, 7, 110, and 12*a*, gives

$$f_s = \frac{124\,678 \times 229}{256 \times 0.89 \times 14 \times 0.15 \times 8} = 7\,440 \text{ lb.}$$

There seems to be some discrepancy between the table of numerical values and the plotted maximum stress at Point 12*a*. In any case, 12*a* shows abnormally large readings compared with the others, but the computed value just found is greater than three of the observed values and greater than the mean of all four.

The diameter of the cap being $0.24L$, the clear span between the edges of the caps on the side belt is $0.76L$, and the least computed distance of the points of inflection along the middle of the side belt is this divided by $\sqrt{3}$, or $0.577 \times 0.76L = 0.44L$; and, consequently, the distance between the points of inflection across the column center is $0.56L$, which, as stated by Mr. Lord in a second paper,* is precisely the observed position of the points of inflection in the side belt of this slab.

Applying Equation (*a*) to the computation of the unit stress at Point 205, Fig. 3, of Mr. Lord's paper, on the edge of the long side belt, gives

$$f_s = \frac{124\,678 \times 229 \times 453}{800 \times 7.6 \times 13 \times 0.15 \times 224}\left[\frac{3 \times 52\,441}{30\,625} - 1\right] = 20\,150 \text{ lb.}$$

per sq. in.

The value observed at 205 was 20 000 lb., and slightly less at the symmetrical position, 208, also situated at the edge of a short side

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* Also published in *Proceedings*, Nat. Assoc. of Cement Users, Vol. VII, 1911.

belt. The corresponding unit stress in a rod tangential to the edge of the cap at 206 was less, for the reason previously stated, being only 14 400 lb. at 206, and 12 300 lb. at 209.

Applying Equation (*b*) to the computation of the limiting unit stress in the rods at the edge of the diagonal belt opposite the column center gives

$$f_s = \frac{124\,678 \times 229}{400 \times 7.6 \times 13 \times 0.15}\left[\frac{3 \times 111\,642}{70\,756} - 1\right] = 22\,440 \text{ lb. per sq. in.}$$

All the observed stresses were somewhat less than 16 800 lb., both at 202, the outside rod, and at 204, the rod tangent to the cap, and 18 300, at 203, the rod lying between them. The outside rod, for the reason already stated, did not show quite the computed stress, but the observed stress in the center rod of the belt, where it crossed the edge of the cap at 207, was 23 400 lb., when all the panels were loaded, and 24 200 lb. when half the load had been removed from the outer panels.

Applying Equation (71) to the computation of the deflections at the center of the panel gives

$$D_2 = \frac{124\,678 \times 224 \times 229 \times 229}{4.4 \times 10^{10} \times 64 \times 14 \times 0.15} = 0.248 \text{ in.}$$

Point 37, Fig. 3,* was the only center point observed in a loaded panel wholly surrounded by loaded panels. The mean value of seven readings was 0.224. When half the load was removed from the outside panels the deflection increased to 0.246 in. The mean readings at the centers of adjacent panels were: Point 4, 0.291 in.; Point 8, 0.306 in.; Point 12, 0.271 in.; all these being somewhat larger because adjacent panels were not loaded.

Applying Equations (54), (58), and (61) to the computation of the deflections at the middle points of the side belts gives $D_1 = 0.1555$ for the long side, and $D_1 = 0.1456$ for the short side. The only reading at the middle point of a long side belt not continuous with or contiguous to an outside panel was at Point 17 where the mean of seven readings was 0.1387. The mean reading at Point 5, at the middle of the continuation of the same rod on the other side of Column 41, between two loaded panels, was 0.229. These points were situated similarly to the middle points of three successive equally loaded spans

* Of Mr. Lord's paper.

of a continuous beam, Point 17 being at the middle point of the middle span, and Point 5 at the middle of one of the end spans; the first deflection was somewhat less, and the other somewhat greater than the deflection when many spans were equally loaded. Point 41 was situated so that its mean deflection should fall between that at 17 and that at 5, which is the fact.

Point 38 was on a short side, on a location in the middle span of three spans like that of 17 on the long side, and the mean reading was 0.127; Point 15 was on a short side, as 5 was on a long side, in an end span of three spans, and the mean reading was 0.163. The computed deflection falls between these, as it should. The deflection at 42 should fall between the readings for Points 38 and 15, which is the fact. Point 11 was in practically the same relative position as 15, and had equal deflections. Other deflections at the middle of outside belts had about half the preceding deflections, influenced more or less by their continuities.

The interaction of successive spans across the column heads, by reason of continuity of spans, is evidently of considerable amount in this floor. This is doubtless due to lack of stiffness at the junction between columns and slab, which in this design is almost entirely without stiffening rods, such as the elbow rods in the mushroom system. The very striking agreement of these computations with the observations in this test show that in all essential particulars the action of the reinforcement is in principle identical with that for which the equations here applied were originally developed.

The Larkin Building.

A very extensive and painstaking test of the Larkin Building, in Chicago, was made by Mr. A. R. Lord.* The design of the slab reinforcement in this building was of the general four-way type used in the mushroom system, but the column heads omitted much of the steel ordinarily used for stiffening the head, and, in place of it, resistance to bending and shear was supplied by a depressed head or "drop" of concrete, 8 ft. square and 6.75 in. thick, placed at the top of each column, between the cap and the bottom of the slab. This was made integral with the slab and formed part of it, just above the cap. The panels are 20 ft. by 24 ft. 2 in., or $L_1 = 290$ in., and

* The results were presented by Mr. Lord in a paper at the Ninth General Convention of Cement Users. Extracts from this appear in the *Cement Era*, January, 1913, p. 52.

$L_2 = 240$ in. The thickness of the slab proper is 9 in., except at the drop, where it is 15.75 in. The diameter of the cap is 60 in. The width of the side belts is from 90 to 93 in., and of the diagonal belts from 105 to 108 in. All the slab rods are ½-in. round, 13 in each short side belt, 22 in each long side belt, and 21 in each diagonal belt.

The floor was designed for a dead load of about 120 lb. per sq. ft., and a live load of from 225 to 250 lb., with a maximum test load of twice the sum of these, or actually 738 lb. per sq. ft. As the dead weight of the slab was acting at all times, the stresses tabulated in the report are those due to the observed elongations plus corrections due to the weight of the slab itself, which is included in the 738 lb. The deflections given, however, are those actually observed under the loads applied, or a maximum test load for deflections of about twice 250 lb., plus 120 lb., or a total actual maximum load of 618 lb. per sq. ft. Hence the total maximum load producing stress was $W = 738 \times 20 \times 24\frac{1}{6} = 356\,700$ lb. The load was applied, and the readings of stresses and deflections were made at several stages. At Stage 2, 130 lb. per sq. ft. was applied over a test area of five panels, three of them wall panels, and two adjacent to them, not wall panels. In Stage 3 the load on this area was increased to 250 lb. In Stage 4, only three panels were loaded, two of them wall panels and one adjacent interior panel, with 415 lb. per sq. ft. Finally, in Stage 5, the entire load of 618 lb. per sq. ft. was placed on two panels only, one wall panel, and an adjacent interior panel, and these were situated so that their short sides were contiguous.

Apply Equation (34) to the computation of the unit stress in the rods of this short side belt, in which, ordinarily, it would be found that $d = 8.25$ in., but among the test data this is stated to have a value of 8 in. in this case. Hence

$$f_s = \frac{356\,700 \times 240}{175 \times 8 \times 13 \times 0.196} = 24\,000 \text{ lb. per sq. in.}$$

The observed stress was 24 200 lb., which would be decreased somewhat, as noted previously, if adjacent panels were also equally loaded, as contemplated in Equation (34). Applying Equation (34) to the long side belt,

$$f_s = \frac{356\,700 \times 290}{175 \times 8 \times 22 \times 0.196} = 17\,000 \text{ lb. per sq. in.}$$

The observed stresses in the rods of a belt lying between a loaded panel and one not loaded is much less than between two loaded panels. The observed value in this location is given at 10 200 lb., but it is not stated whether this is a mean value or the greatest value for any rod of the belt, or the stress in the middle rod. From the figured location of the test lines, it is probably the latter. The computed results should be regarded as in most satisfactory agreement with the observations.

Applying Equation (52) to the computation of the unit stress in the middle rod of the diagonal belt at the center of the panel:

$$f_s = \frac{356\,700 \times 290}{256 \times 0.9 \times 7.75 \times 21 \times 0.196} = 14\,070 \text{ lb. per sq. in.}$$

in which 7.75 in. is taken as the mean distance of the center of the steel from the upper surface at the center of the panel. The given observed value was 12 900 lb., which differs from the computed by less than 10 per cent. It is not stated whether the observation was in the wall panel or in the interior panel, or whether it was an average or in the middle rod of the belt, but it was probably the latter. If so, the stress in the middle rod should be greatest. This result may be regarded as satisfactory, and very near what would be observed in the middle rod if the adjacent panels were equally loaded.

Before computing the stresses in the head steel at the edge of the cap and edge of the drop, it should be noted that all the stresses given in the general summary of stresses for the maximum load of 738 lb. are from 2½ to 3 times as great as for the design load, which is half as great. This, among other things, signifies that the value of the modulus of elasticity for the concrete was much less under the test load than was its initial value. Under the test load of 738 lb., there is no one of the columns entirely surrounded by loaded panels. The consequence is that the observed stresses in all the belts of the head will be much less than those computed, and if an attempt were made to compute the stresses due to the design load which was spread over all the panels around the column, where most of the readings on the head belts were evidently made, the observed stresses would have to be multiplied by a coefficient of 1.5 or more to make them comparable with the results at the test load, 738 lb.

Now write Equation (*a*) in the general form

$$f_s = \frac{W\,L_1\,(L_1 + L_2)}{800\,d_3\,A_1\,L_2}\left[\frac{3\,(2\,x)^2}{B_1^{\,2}} - 1\right]\ldots\ldots\ldots\ldots(a)$$

in order to compute the unit stress in the rods of the long side belt at the edge of the cap, where $2x = B_1 = 290 - 60 = 230$ in., and $d_3 = 14$ in. The drop is so thick as to obviate mostly the increase of stress in the center rod where it crosses the edge of the cap, and therefore no account is taken of such increase. This unit stress is found to be $f_s = 9\,460$ lb. per sq. in. The observed stress along this side at the edge of the load, of course, was considerably smaller, and is given as 7 000 lb.

Making a similar computation at the edge of the drop on this belt, where $B_1 = 230$ in., $2x = 290 - 96 = 194$ in., and $d_3 = 7$ in., it is found that $f_s = 14\,500$ lb. per sq. in. Along this side belt at the edge of the load the observed result was only 10 400. The diagram of gauge lines seems to show that these are readings observed along the middle rod of the belt. In that case the outside rod at the edge of the belt under a load over many panels would have shown greater unit stresses than those computed, or, in the writer's opinion, at least as large.

Applying Equation (*a*) to the computation of the unit stresses in the short side belt at the edge of the cap, by exchanging the subscripts, 1 and 2, and placing $2x = B_2 = 240 - 60 = 180$ in., $d_3 = 14$ in., and $A_2 = 13 \times 0.196$, gives $f_s = 10\,950$ lb. per sq. in. This belt crosses the cap from the unloaded into the loaded area which covers the two panels on each side of it. The tipping of the head relieves to some extent the stresses which would exist were all panels loaded, and the reading given is 7 300 lb.

Again, compute the unit stress at the edge of the drop in this short side belt by inserting in the equation just used $B_2 = 180$ in., $2x = 240 - 96 = 144$ in., $d_3 = 7$ in., and $f_s = 15\,300$ lb. per sq. in. At this point there is something abnormal and inexplicable in the observed results which, apparently, should be larger than those observed at the edge of the cap where the concrete is so thick. This contradictory reading is only 5 800 lb., and may be due to some bend in the rod, or other peculiarity, because the same thing appears at the smaller loads.

Writing Equation (b) in the general form

$$f_s = \frac{C_1\, W\, L_1}{400\, d_3\, A}\left[\frac{3\,(2\,x')^2}{B'^2} - 1\right] \dots\dots\dots\dots\dots (b)$$

and computing the unit stress in the middle rod at the edge of the cap on the diagonal of 21 rods by increasing B' between the corners of the assumed square caps and placing

$$L = \sqrt{290^2 - 240^2} = 320 \text{ in., } B' = 320 - 85 = 235 \text{ in.,}$$

$$2x' = \quad 320 - 60 = 260 \text{ in., } f_s \text{ is found to be as follows:}$$

$f_s = 16\,160$ lb. per sq. in. The smaller observed value was 14 200 lb. 10 800 lb.

Apply the same equation to compute the unit stress in the middle rod of the diagonal belt at the edge of the drop, assumed to be 96 in. in diameter because the projecting corner should be disregarded, and we have $2x' = 320 - 96$ in., $B' = 235$ in., $d_3 = 7$ in. Hence $f_s = 16\,160$ lb. per sq. in. The smaller observed value was 14 200 lb.

Applying Equation (70) to compute the deflection at the center of the panel gives

$$D_2 = \frac{W\, L_2\, L_1^2}{10^{10}\, A_1}\left[\frac{1}{6.56\, d_2^2} + \frac{1}{12.5\, d_3^2}\right]$$

in which the total load producing the observed deflection was 618 lb. per sq. ft., or a total load of $W = 296\,850$ lb.

$$D_2 = \frac{296\,850 \times 240 \times 290^2}{10^{10} \times 22 \times 0.196}\left[\frac{1}{6.56 \times 7.75^2} + \frac{1}{12.5 \times 14^2}\right]$$

or $D_2 = 0.41$ in. The observed deflections at Points 10 and 16 were 0.40 in., an extraordinary agreement, under the circumstances, of reduction of steel in the head and displacement of inflection lines which produce effects that tend to balance each other.

Deflections at the middle of the long side may be computed by Equations (91), (54), and (58), as follows:

$$D_1 = \frac{296\,850 \times 290^3}{10^{10} \times 22 \times 0.196}\ \frac{1}{10.7 \times 8^2}\ \frac{2.2}{60 \times 14^2} = 0.276 \text{ in.}$$

The observed deflections were 0.21 in. at Point 15 and 0.26 in. at Point 17, on the edges of the loaded area, which values would be somewhat increased if the adjacent panels were also equally loaded. A similar computation at the middle of a short side belt gives results still more in excess of the value observed at Point 13, Fig. 3, of Mr. Lord's paper,

by reason of the shortness of the concave section between the points of inflection. The surprising thing is, not that such discrepancies exist between computed and observed results, but that equations derived for a flat slab differing so widely and in so many particulars from this one should nevertheless be applicable at all.

The writer's equations were derived for the case of side belts with total cross-sections proportional to their spans and with diagonals with intermediate cross-sections which, according to his theory, is the correct proportion. The Larkin slab, however, has the cross-sections of its side belts proportional to the cubes of their spans, which is a highly uneconomical arrangement of steel, as shown by the unit stresses observed in the test. The fact that these equations apply so closely shows that the use of the drop in stiffening the head is practically identical in action with the steel frame of the column head which it replaces.

In view of the unqualified assertion made in the extract from Mr. Lord's paper in the *Cement Era,* respecting the Larkin test, that "the deflection readings, compared with those of other tests, show that the resulting structure is stiffer and stronger than the straight flat slab as ordinarily designed, with anywhere the same amount of materials", it becomes of interest to submit such a straight flat slab to analysis for purposes of comparison, as the writer's formulas are applicable to such a slab with considerable accuracy.

It is to be noticed that deflection readings do not of themselves permit any conclusions whatever to be drawn as to the strength of a structure, but have reference to stiffness only, so that that half of the assertion referring to strength is, on the face of it, unwarranted and entirely unsupported by these readings, and, in fact, will be found to be incorrect, as the writer will proceed to show. As to deflections, it may be that a perfectly flat slab might have a deflection of perhaps not more than 10% in excess of this slab, with its bulky and unsightly drop at the head of each column. If that should possibly be the case, it is a trivial matter, and of no consequence compared with the very serious loss of strength involved in this construction.

The volume of the drop, which is 8 ft. square and 6.75 in. thick, is 62 200 cu. in. In case this quantity of concrete were used to increase the mean thickness of the slab, it would make a flat slab approximately 0.9 in. thicker than at present. Such a flat slab 9.9 in. thick through-

out, with belts of 0.5-in. round slab rods, having 19 rods on each short side, 19 in each diagonal belt, and 22 on each long side, would have slightly less slab steel, and the same quantity of concrete in the slab as in the Larkin Building, but the maximum stresses in the steel would be far less.

The unit stress in the slab rods at the middle of the long side belt of this straight flat slab is found by Equation (34) to be

$$f_s = \frac{WL_1}{175\, d_1\, A_1} = \frac{356\,700 \times 290}{175 \times 9 \times 22 \times 0.196} = 15\,230 \text{ lb. per sq. in.}$$

Similarly, the unit stress at the middle of the slab rods in the short side belt is

$$f_s = \frac{WL_2}{175\, d_1\, A_2} = \frac{356\,700 \times 240}{175 \times 9 \times 19 \times 0.196} = 14\,600 \text{ lb. per sq. in.}$$

By Equation (52) the unit stress in the diagonals at the center of the panel is

$$f_s = \frac{W\,L_1}{256\, j\, d_2\, A_2} = \frac{356\,700 \times 290}{256 \times 0.89 \times 8.56 \times 19 \times 0.196} = 14\,400 \text{ lb. per sq. in.}$$

It would require a load of more than 1 200 lb. per sq. ft. on such a mushroom slab to produce a maximum unit stress on the slab rods as great as was caused in the Larkin slab by the test load of 738 lb. per sq. ft. The difference is so great as to nullify entirely any such unwarranted claims as those quoted in the extract from the *Cement Era*.

It thus appears that a mushroom slab more than 65% stronger than the slab in the Larkin Building can be constructed at the comparatively slight additional expense of part of the steel required for the radial and ring rods of the mushroom head. How much more steel might be required for the mushroom is unknown, because no data are available as to the quantity of steel used in the top rods which were embedded in the top of the slab over the column heads, and in the frame supporting the slab rods in the column heads in the Larkin slab. In Mr. Lord's paper a photograph of the steel in place for pouring the concrete shows a rectangular frame support of this kind. Any claim for superiority of the Larkin structure with reference to strength and economy of materials over a properly designed "straight" flat slab is, in the writer's opinion, wholly without foundation.

THE DONNELLY BUILDING.

The Donnelly Building, in Chicago, Ill., was designed and tested by Mr. A. R. Lord. Certain data of this test, as yet unpublished, have been put at the writer's disposal by C. A. P. Turner, M. Am. Soc. C. E. The panels in the slabs are 24 ft. 10 in. by 24 ft. 4 in. and 11 in. thick, with a drop of concrete at the head of each column, 9 ft. square and 9 in. thick, integral with the slab. The total thickness of the slab at the edge of the cap is 20 in.; the effective thickness from the center of the reinforcement in the head to the lower surface of the slab is 18 in.

Design load, 300 lb. per sq. ft.;
Weight of slab, 150 lb. per sq. ft.;
Test load, 750 lb. per sq. ft.;
Area of panel, 604.3 sq. ft.;
$L_1 = 298$ in.;
$L_2 = 292$ in.

All the reinforcement was of $\frac{1}{2}$-in. round rods. Each long side belt had 26 rods, and each short side belt and diagonal belt, 25 rods. The computed maximum unit stress at the center of the short side belt, by Equation (34), is

$$f_s = \frac{292\,W}{175 \times 25 \times 0.196 \times 10.25}.$$

For a working load of $300 + 150 = 450$ lb. per sq. ft., $f_s = 9\,000$ lb. For the total test load of double this quantity, that is, 750 lb. per sq. ft., in addition to the weight of the slab itself, $f_s = 18\,000$ lb. when all the panels are equally loaded.

The deflection at the center of the panel may be computed for the test load of 750 lb. per sq. ft. by Equation (70) as follows, by assuming $\Sigma A = 6A_1$ as the reinforcement in the area over the head of the columns, this being 80% of that assumed in the standard mushroom design:

$$D_2 = \frac{604.3 \times 750 \times 292 \times 298^2}{10^{10} \times 25 \times 0.196}\left[\frac{1}{6.56 \times 9.75^2} + \frac{1}{10 \times 18^2}\right] = 0.384 \text{ in.}$$

The observed deflection was barely 0.3 in., which shows that the lines of inflection in this design were somewhat nearer the center of the panel than assumed in Equation (70), as they were also in the Larkin Building.

If a straight mushroom slab be designed with the same quantity of concrete in it as the Donnelly slab, the drop will furnish sufficient concrete to make the slab 1.2 in. thicker, giving a total thickness of 12.2 in. The computed stress at the middle of the side belts, in case all the slab belts remain unchanged, will be $f_s = 16\,100$ lb., instead of 18 000 lb. under the test load, and the computed deflection will be barely 0.4 in. at the center of the panel.

The Donnelly slab has a 9½-ft. square frame of 1⅛-in. round rods to support the slab rods in each head, which, with the lap, is more than 40 ft. long. A standard mushroom head would require more steel than this for radial and ring rods, but the quantity is very small compared with its considerable increase of strength, and this without increasing the deflection beyond a very conservative and in every way permissible value. Therefore, a straight flat slab design, such as the mushroom, appears to be preferable to one with a large drop like this. Means for making a rational comparison not having been available heretofore, it has not been possible to state positively that the unwarranted claims that have been made for designs with a large drop were unfounded; but that such is the fact is now demonstrable, as appears from the foregoing computations.

The St. Paul Bread Company Building.

The floor of the St. Paul Bread Company Building is a rough slab, 6 in. thick, and has 16 by 15-ft. panels, reinforced with ⅜-in. round rods, ten in each belt. The design load was 100 lb. per sq. ft. The slab, Fig. 19, was constructed in winter and frozen, but, as the final test was postponed until August, 1912, the slab was very fully cured, considerably more so, in fact, than most slabs when subjected to test. The test was made by Professor W. H. Kavanaugh, of the University of Minnesota, in the following manner:

First, extensometer measurements were made on seventeen 8-in. lengths of slab rods, which were exposed under a single loaded panel, three of these, Nos. 1, 2, and 3, being at the middle of three rods of one long side belt at the edge of the load, and the remaining fourteen distributed over the central area of one diagonal belt. Second, measurements of deflections were made at two points, one at the center of the panel and one near the middle of the interior edge of one long side belt. Third, the embedment of the rods was tested. Table 2 con-

tains the observed elongations due to change of loading at all the seventeen positions for each of the test loads, 108.4, 316.8, and 416.8 lb. per sq. ft., as well as after the removal of the load. A comparison of the observed elongations at symmetrical points reveals such discrepancies in the observations as to require some preliminary discussion.

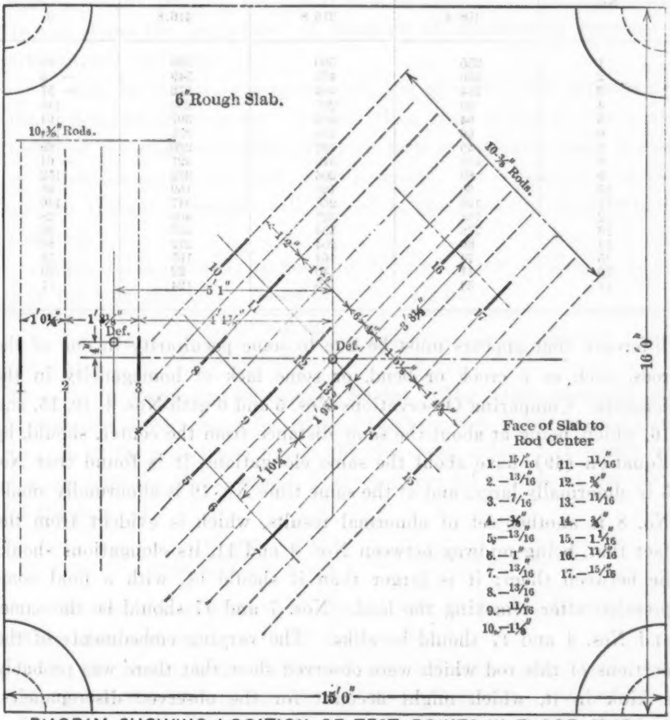

DIAGRAM SHOWING LOCATION OF TEST POINTS IN FLOOR SLAB.
SAINT PAUL BREAD COMPANY BUILDING.
"TURNER MUSHROOM SYSTEM"

Fig. 19.

Observations Nos. 5 and 6 were on a pair of diagonal rods on each side of and adjacent to the diagonal line of the panel, there being no rod exactly on the diagonal, and situated just beyond the edge of the other diagonal belt. No reason can be discerned for any difference between these elongations, but the wide

TABLE 2.—Observed Elongations, in Inches per Million Inches, Under Given Loads per Square Foot.

Note.—Obtain actual unit stresses by multiplying by 30.

Observation No.	Elongations under the Following Loads, in Pounds.			
	108.4	316.8	416.8	0
1	255	500	598	80
2	236	473	549	− 3
3	244	464	572	− 57
4	63	207	233	145
5	64	262	397	164
6	66	168	271	114
7	45	152	220	69
8	218	370	421	− 64
9	89	266	372	152
10	68	120	159	46
11	122	263	347	146
12	153	327	400	28
13	276	534	653	93
14	60	204	272	80
15	74	164	165	18
16	11	40	22	− 30
17	34	70	124	17

difference that appears must be due to some peculiarity in one of the rods, such as a crook or bend, or some lack of homogeneity in the concrete. Comparing Observations Nos. 5 and 6 with Nos. 4, 10, 15, and 16, which, being at about the same distance from the center, should, by Equation (49), have about the same elongations, it is found that No. 5 is abnormally large, and at the same time No. 16 is abnormally small. No. 8 is another set of abnormal results, which is evident from the fact that, being midway between Nos. 4 and 11, its elongations should lie between them; it is larger than it should be, with a final compression after removing the load. Nos. 7 and 17 should be the same, and Nos. 4 and 17 should be alike. The varying embedments of the portions of this rod which were observed show that there was probably a kink in it, which might account for the observed discrepancies. It is possible, however, that some such differences may appear when the loading is piled on one side of the panel before piling it on the other. No such explanation, however, will fit the case of Nos. 12 and 13, which are at the middle of the two rods adjacent to the diagonal line at the center of the panel. There appears to be no question that No. 13 is abnormally large, for No. 12 agrees well with others, being only a little larger than those at the nearby positions 9, 11, and 14, and the values at No. 13 are in wide disagreement with them. The

very considerable differences between results which should apparently be equal makes it evident how inexact single determinations must often be by reason of bends in the rods, lack of homogeneity in the concrete, etc., and emphasizes the importance of carefully laying belt rods straight and having them spaced uniformly, as well as embedded equally, before pouring the concrete, if consistent results are desired. It also shows the importance of checking all readings by readings at symmetrical positions.

It may be stated in general that the observed unit stresses and the deflections in this test are less than they would be for a slab tested at the stage of curing at which tests are usually made, a stage to which the equations apply more precisely. In consequence of this, all the computed results will exceed to some extent those actually observed.

Apply Equation (34) to compute the unit stress at Nos. 1, 2, and 3 of the long side belt. Assuming $d_1 = 5.3$ in.,

$$f_s = \frac{100\,000 \times 192}{175 \times 5.3 \times 1.1} = 18\,800 \text{ lb. per sq. in.}$$

in case of many panels equally loaded. The mean observed unit stresses for three rods at the edge of the load was 17 190 lb., the stress in each of the rods being practically the same, a fact that speaks well for the stiffness of the head. The computed unit stress at the center of the panel is

$$f_s = \frac{100\,000 \times 192}{256 \times 0.9 \times 5 \times 1.1} = 15\,150 \text{ lb. per sq. in.}$$

The observed unit stress at No. 12 was 12 000 lb., though at No. 13 the abnormal value of 19 600 lb. was found.

As the observations of stresses and deflections were made when only a single panel was loaded, and the computations assume that all the panels are equally loaded, any very close agreement of Table 2 with the observed results is not to be expected; nevertheless, comparison with that table shows that the computed results agree with the observations far better than the observations agree among themselves at such symmetrical points as admit of any comparisons.

The deflection at the center of the panel under a load of more than double the design load plus once the dead load, namely, 316.8 lb. per sq. ft., was 0.32 in., which is less than $\frac{1}{800}$ of the diagonal span. To

compute the deflections at the panel center, apply Equation (71), as follows:

$$D_2 = \frac{100\,000 \times 180 \times 192^2}{4.4 \times 10^{10} \times 5 \times 1.1} = 0.56 \text{ in.}$$

Computation of the deflection at the point near the middle of the interior edge of a long side belt gives a deflection for a load of 100 000 lb. on each panel of approximately 0.4 in.

TABLE 3.—DEFLECTIONS, IN INCHES,
UNDER GIVEN LOADS PER SQUARE FOOT.

	Test Loads, in Pounds.			
	108.4	316.8	416.8	0
Center of panel..................Observed	0.077	0.320	0.437	0.155
" " " ....Computed	0.146	0.426	0.560	.....
Edge of side belt.................Observed	0.065	0.247	0.332	0.124
" " " " ....Computed	0.104	0.304	0.400	.....

As before stated, the somewhat large excess of computed over observed deflections is due to two circumstances: first, and principally, to the age and consequent stiffness of the slab; and secondly, at the edge, to the fact of a single panel load instead of many, neither of which circumstances is taken account of in the equations used.

THE STUDEBAKER AUTOMOBILE BUILDING.

The Studebaker Automobile Building, Michigan Avenue and 21st Street, Chicago, Ill., was erected in 1910, and is an example of a system of construction with two-way reinforcement only. The distances between column centers are 24 ft. and 23 ft. 9 in. Papers descriptive of this system may be found in the current technical journals.* All these papers, except the second, were written by T. L. Condron, M. Am. Soc. C. E., whose company designed the Studebaker Building. The last paper gives the results of tests of this and other buildings built on this system, while the second and third papers give the sizes and quantities of steel reinforcement in this particular building, from which it appears that the weight of steel reinforcement per panel was 2 600 lb.

Two kinds of floors are constructed on this two-way system; one is a perfectly flat slab of uniform thickness throughout; the other has

*Journal, Western Society of Engineers, December, 1909; *Engineering Record*, January 22d, 1910; *Engineering News*, January 27th, 1910; *Western Contractor*, March 12th, 1913.

a large so-called "panel" in the ceiling in the middle of the space between each four columns. The Studebaker Building is of this second kind, so that in the middle of each panel proper, 24 ft. by 23 ft. 9 in., there is this architectural "panel" about 18 ft. square. The thicker parts of the slab, extending from column head to column head, thus constitute shallow girders which in this case are 12 in. thick and about 6 ft. wide; and the central panels, 18 ft. square, are only 6 in. thick, giving a mean thickness of slab of 8.6 in., if the same quantity of concrete were uniformly distributed.

The design load was 100 lb. per sq. ft., and the dead load of the slab itself was also assumed at 100 lb. per sq. ft. The test load was applied on a single panel, and, in accordance with the Chicago Ordinances, amounted to 300 lb. per sq. ft., or a total of 300 $\times$ 570 = 171 000 lb. This caused a deflection at the center of 0.34 in.

This floor will be compared with a mushroom slab, 8.6 in. thick, designed to carry the same loading.

The Studebaker slab had a thickness of 1 in. of concrete clear under the $\frac{3}{4}$-in. square girder rods, and $\frac{3}{4}$ in. clear of concrete under the $\frac{1}{2}$-in. round rods at the bottom of the central panel. The mushroom slab will be assumed to have twenty-five $\frac{5}{8}$-in. round rods in each belt, and 0.6 in. of concrete clear under the rods at the bottom of the slab, it being customary to make this fire-proofing layer less for smaller rods, and in any case not less than 0.5 in.

By Equation (34) the stress in the side belt under the dead load of 100 lb., plus the test load of 300 lb. per sq. ft., will be

$$f_s = \frac{228\,000 \times 288}{175 \times 7.8 \times 2.76} = 17\,400 \text{ lb. per sq. in.}$$

Mr. Condron states that he has used 17 000 lb. as the unit stress in the steel. Under a working load consisting of 100 lb. dead load plus 100 lb. design load the stress would be one-half the value computed above, namely, 8 700 lb. per sq. in.

The quantity of slab steel per panel in this design depends somewhat on the length of lap in the splicing. Without any lap, there would be in each 24 by 23.75-ft. panel:

Side rods,	25×24	= 600	ft.
Side rods,	25×23.75	= 594	"
Diagonal rods,	50×33.75	= 1 687.5	"

Total length of $\frac{5}{8}$-in. round rods = 2 881.5 ft.

The weight of this is $2\,881.5 \times 376 = 1\,081.5$ lb. In round numbers, it will be assumed that the steel in the mushroom head, consisting of radial and ring rods, will bring the total weight up to 1 300 lb. per panel, which is just one-half that in the Studebaker Building with the same quantity of concrete.

Now, as to deflections: by Equation (71), the central deflection due to the test load is

$$D_2 = \frac{171\,000 \times (288)^3}{4.4 \times 10^{10} \times 2.76 \times (7.4375)^2} = 0.594 \text{ in.}$$

If the cross-section of the steel were doubled throughout, the deflection would be one-half of this, or 0.297 in., which is only slightly less than 0.34 in., which was the central deflection observed in the Studebaker test.

It will be noted that the formulas in the writer's investigations are dependent on the quantity of reinforcement, and not on the directions in which it runs, the only condition being that the belts shall cross in such immediate proximity to each other as to make their mutual action on each other effective. This does not occur with perfect uniformity everywhere in the two-way system under consideration, and, consequently, it would be expected to show somewhat greater deflections with the same quantity of steel than would the mushroom system.

The special claim made for this system by its designers is that in it the stresses in the steel reinforcement can be readily computed by the method of moments due to the loading. No tests have been published, so far as the writer knows, to substantiate this claim, and in his judgment such tests would show a difference, between extensometer observations and those obtained by the formulas published in the papers previously referred to, of not less than 50% of the observed amount, and probably more nearly 100 per cent.

This conclusion is based not only on the general theory of such structures, with which the writer has some familiarity, but on the computations just given in this paper, which fairly support this statement.

APPENDIX A.

EXTRACTS FROM REPORT OF TEST OF THE NORTHWESTERN GLASS COMPANY BUILDING,*

BY F. R. McMILLAN.

Description of the Building.—This is a four-story-and-basement warehouse, of the skeleton type of reinforced concrete construction, erected for the Northwestern Glass Company, of Minneapolis, from plans prepared by Bertrand and Chamberlin, Architects, and Mr. C. A. P. Turner, Consulting Engineer. In plan it is 66 by 163 ft., divided into 16 by 17-ft. panels. The floor is of the "Mushroom" type of flat slab construction, designed for a live load of 400 lb. per sq. ft. The 8-in. rough slab is reinforced with fifteen ⅜-in. round rods in each direct and diagonal belt, as shown on Fig. 1. At the time of the test, the 2-in. cement finish which is to form the wearing surface was not in place.

Age and Conditions of Curing.—At the time of the test, the concrete in the test panels had been in place 5 months, but the weather had not been favorable for curing, except during the last few weeks. On the day the test panels were cast, December 6th, 1912, the mean temperature was 10°, with a maximum of 19° and a minimum of 0° Fahr. From this time until the end of March, 1913, the temperature practically remained below freezing. The usual precautions were taken to prevent freezing of the green concrete. Salamanders were kept burning in the basement until January 10th. The slab was kept covered with boards and canvas from the day it was poured until the erection of the forms for the second floor was begun. This interval was about a week or 10 days. Pouring of the second floor was begun on December 28th, and from this time until January 10th salamanders were kept fired under the second floor. The forms supporting the test panels were removed near the end of February. The last few weeks before the test the weather was very mild, and it is said that the concrete showed a noticeable increase in hardness during this period. At the time of the test, May 12th to 19th, the floor had all the appearance of an excellent hard concrete.

Plan and Description of the Test.—As the time available for the test, and the floor space that could be occupied in the basement, were limited by the necessity of caring for large shipments of glass, attention was confined to the measurement of deflections and deformations at critical points only. Very little choice was had in selecting points on the upper side of the slab at which to determine steel extensions;

* Figs. 1 to 14, inclusive, are also taken from this report.

for, with the type of extensometer used, measurements could only be taken on the top belt of rods and at points very near the surface.

Extensometer measurements were taken at various stages during the loading and unloading, and in all but one instance they were taken immediately after each load was in place and again just before applying the next load. Deflections were taken at a number of intermediate stages at which no extensometer measurements were made. The extensometer measurements were made with the regular short-legged laboratory form of Berry strain gauge, reading over an 8-in. gauge line. The deflections were measured with a portable instrument specially designed to enable measurements to be taken at a large number of points without obstructing passage under the various panels.

The test load was of cement in bags, arranged in four piles in each panel, separated by aisles to prevent arching and to facilitate the taking of observations. The entire four panels were first loaded uniformly in three stages up to a total load of 540 lb. per sq. ft. of loaded area. The load from two panels was then nearly all transferred, in two stages, to the remaining two panels, making an intensity of 1 000 lb. and 750 lb. per sq. ft. on the loaded areas of these two panels. The removal of the load was made by stages in such a manner that the one panel retained its full load of 1 000 lb. per sq. ft. for 68 hours. The detail of areas and intensities of loading are shown on Figs. 5 and 6.

The accompanying tables and drawings show all the data of loading and the resulting steel stresses and deflections. These are self-explanatory. The steel stresses have been calculated from the extensometer measurements on a basis of 30 000 000 lb. per sq. in. for the modulus of elasticity. Fig. 13 shows the actual thickness of the test slabs and the depth of the rods at the top on which extensometer measurements were made.

TABLE 4.—Time Schedule of Loading.

Test of Northwestern Glass Company Building.

Date, 1913.	Load No.	Applying Load.			Interval, Load in Place.	
		Began.	Noon.	Finished.	Hr.	Min.
May 13.....	1	10:15 A. M.	12-12:30	1:10 P. M.	....	....
May 14.....	2	9:30 A. M.	12-12:30	2:45 P. M.	25	35
May 15.....	3	10:30 A. M.	12-12:30	1:50 P. M.	23	5
May 16.....	4	12:45 A. M.	....	3:00 P. M.	25	10
May 17.....	4a	11:00 A. M.	12-12:30	3:00 P. M.	24	0
May 17.....	5	3:45 P. M.	....	4:45 P. M.	1	45
May 18.....	6	9:45 A. M.	....	10:40 A. M.	17	55
May 18.....	7	11:15 A. M.	....	12:00 NOON.	1	20
May 19.....	8	10:30 A. M.	12-12:30	12:53 P. M.	24	53
May 19.....	9	1:30 P. M.	....	4:00 P. M.	3	7

TABLE 5.—Summary of Loading.

Load No.	Total Panel Loads.				Load per Square Foot Over Loaded Area.			
	A	B	C	D	A	B	C	D
1	31 540	31 445	29 355	29 070	181	178	169	162
2	68 780	67 830	68 875	66 025	395	385	396	364
3	96 045	93 955	97 090	92 910	552	533	558	512
4	59 470	130 815	58 995	130 720	342	742	339	720
4a	59 470	130 815	6 745	182 970	Same.	Same.	39	1 004
5	None.	132 430	Same.	Same.	None.	751	Same.	Same.
6	"	60 135	"	"	"	341	"	"
7	"	None.	"	"	"	None.	"	"
8	"	"	"	94 715	"	"	"	521
9	"	7 000	7 000	None.	"	40	40	None.

TABLE 6.—Loading Data, Panel A.

Panel, 16 by 17 ft. Area, 272 sq. ft. Loaded area, 174.1 sq. ft.

Load No.	Total Loads in Sub-Divisions:				Total load in panel.	Loads per Square Foot in Sub-Divisions:				Average Load per Square Foot of:	
	A-1.	A-2.	A-3.	A-4.		A-1.	A-2.	A-3.	A-4.	Loaded area.	Whole panel.
1	8 265	9 025	7 315	6 935	31 540	182	192	178	172	181	116
2	17 385	18 525	16 815	16 055	68 780	382	394	408	397	395	253
3	24 130	25 270	23 845	22 800	96 045	531	537	579	565	552	353
4	15 105	16 150	14 440	13 775	59 470	332	344	350	341	342	219
4a	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.
5	None.	None.	None.	None.	None.	None.	None.	None.	None.	None.	None.
6	"	"	"	"	"	"	"	"	"	"	"
7	"	"	"	"	"	"	"	"	"	"	"
8	"	"	"	"	"	"	"	"	"	"	"
9	"	"	"	"	"	"	"	"	"	"	"

TABLE 7.—Loading Data, Panel B.

Panel, 16 by 17 ft. Area, 272 sq. ft. Loaded area, 176.3 sq. ft.

Load No.	Total Loads in Sub-Divisions:				Total load in panel.	Loads per Square Foot in Sub-Divisions:				Average Load per Square Foot of:	
	B-1.	B-2.	B-3.	B-4.		B-1.	B-2.	B-3.	B-4.	Loaded area.	Whole panel.
1	7 600	7 980	8 550	7 315	31 455	169	191	187	167	178	116
2	15 960	17 100	17 955	16 815	67 830	355	409	393	384	385	249
3	22 135	23 845	24 415	23 560	93 955	492	571	535	538	533	345
4	30 875	33 345	33 535	33 060	130 815	686	798	734	755	742	481
4a	Same.	Same.	Same.	Same.	130 815	Same.	Same.	Same.	Same.	Same.	Same.
5	32 490	"	"	"	132 430	722	"	"	"	751	487
6	15 485	14 820	15 390	14 440	60 135	344	355	337	330	341	221
7	None.	None.	None.	None.	None.	None.	None.	None.	None.	None.	None.
8	"	"	"	"	"	"	"	"	"	"	"
9	"	3 500	3 500	"	7 000	"	84	77	"	40	26

TABLE 8.—Loading Data, Panel C.

Panel, 16 by 17 ft. Area, 272 sq. ft. Loaded area, 173.9 sq. ft.

Load No.	Total Loads in Sub-Divisions:				Total load in panel.	Loads per Square Foot in Sub-Divisions:				Average Load per Square Foot of:	
	C-1.	C-2.	C-3.	C-4.		C-1.	C-2.	C-3.	C-4.	Loaded area.	Whole panel.
1	7 885	7 220	6 080	8 170	29 355	157	172	163	183	169	108
2	19 285	16 340	14 820	18 430	68 875	383	390	398	414	396	253
3	27 360	23 085	20 615	26 030	97 090	542	551	554	586	558	357
4	16 435	14 060	12 635	15 865	58 995	326	336	340	356	339	217
4a	4 750	None.	None.	1 995	6 745	94	None.	None.	45	39	25
5	Same.	"	"	Same.	Same.	Same.	"	"	Same.	Same.	Same.
6	"	"	"	"	"	"	"	"	"	"	"
7	"	"	"	"	"	"	"	"	"	"	"
8	"	"	"	"	"	"	"	"	"	"	"
9	None.	3 500	3 500	None.	7 000	None.	83	94	None.	40	26

TABLE 9.—Loading Data, Panel D.

Panel, 16 by 17 ft. Area, 272 sq. ft. Loaded area, 181.6 sq. ft.

Load No.	Total Loads in Sub-Divisions:				Total load in panel.	Loads per Square Foot in Sub-Divisions:				Average Load per Square Foot of:	
	D-1.	D-2.	D-3.	D-4.		D-1.	D-2.	D-3.	D-4.	Loaded area.	Whole panel.
1	7 410	7 315	6 840	7 505	29 070	168	158	159	156	162	107
2	16 150	16 435	16 055	17 385	66 025	365	364	372	362	364	243
3	22 895	22 895	22 515	24 605	92 910	520	494	522	514	512	342
4	32 680	31 825	31 350	34 865	130 720	740	686	728	727	720	480
4a	47 405	45 980	38 760	50 825	182 970	1 073	994	900	1 059	1 004	672
5	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.
6	"	"	"	"	"	"	"	"	"	"	"
7	"	"	"	"	"	"	"	"	"	"	"
8	22 895	22 895	22 515	26 400	94 705	518	494	522	550	521	348
9	None.	None.	None.	None.	None.	None.	None.	None.	None.	None.	None.

TABLE 10.—TEST OF NORTHWESTERN GLASS COMPANY BUILDING.

SLAB DEFLECTIONS. PANELS B AND A.

Deflections are given in thousandths of an inch.

Load No.	Date, May, 1913.	DEFLECTIONS AT POINT NUMBER.											
		1	2	3	28	29	30	4	5	6	7	8	9
Zero.	12th	0	0	0	....	....	....	0	0	0	0	0	0
1	13th	11	22	16	....	....	....	11	15	43	32	34	18
1	14th	19	23	14	....	....	....	11	15	43	28	30	12
2	14th	59	102	60	....	....	....	44	66	105	115	121	56
2	15th	77	121	70	0	0	0	54	76	118	130	141	70
3	15th	130	206	111	86	101	69	89	124	182	219	252	135
3	16th	144	237	131	105	145	98	101	144	211	260	282	147
4	16th	158	260	138	156	251	178	94	131	186	225	241	120
4	17th	169	275	146	173	281	208	92	132	188	226	241	122
5	17th	154	250	132	170	293	211	77	102	141	160	149	71
5	18th	153	248	131	179	310	226	71	96	136	151	140	67
6	18th	....	....	....	108	187	143	....	....	....	....	....	....
7	18th	97	148	78	29	63	50	60	83	121	128	122	56
7	19th	83	139	70	20	55	40	60	82	120	129	127	49
8	19th	....	138	....	19	51	40	....	76	115	121	125	*
9	19th	89	142	74	22	62	*	50	69	105	115	116	....
9	20th	91	145	74	26	54	....	53	69	106	115	119	....
9	24th	82	136	77	17	37	....	47	63	98	104	107	....
10	June 5th	*	128	78	*	*	....	46	64	*	105	108	....

*These points were inaccessible. Note that deflections given for Points 28, 29, and 30 represent deflections below the position of the slab at Load 2. At all other points the deflections shown are from the initial position of the slab.

TABLE 11.—TEST OF NORTHWESTERN GLASS COMPANY BUILDING.

SLAB DEFLECTIONS. PANEL D.

Deflections are given in thousandths of an inch.

Load No.	Date, May, 1913.	DEFLECTIONS AT POINT NUMBER.												
		10	11	12	13	14	15	16	17	18	19	20	21	27
Zero	12th	0	0	0	0	0	0	0	0	0	0	0	0	0
1	13th	15	25	25	32	10	17	32	37	37	27	22	11	0
1	14th	10	19	23	28	6	9	22	32	31	21	16	4	10
2	14th	49	78	104	105	42	55	79	106	109	48	78	33	47
2	15th	52	88	92	111	58	73	99	118	131	72	87	38	55
3	15th	74	124	154	166	72	95	136	168	200	95	89	50	77
3	16th	86	140	170	187	80	106	149	189	191	111	132	54	81
4	16th	92	143	179	235	94	128	183	242	251	147	137	80	89
4	17th	92	148	173	247	101	142	199	264	276	161	147	90	90
4*b*	17th	......	......	......	......	......	......	......	......	333	......	....	....	....
4*a*	17th	......	......	......	......	......	......	......	......	396	......	....	....	....
5	17th	100	164	193	343	140	198	276	387	416	250	172	150	110
5	18th	99	169	208	374	150	214	297	423	456	278	182	160	113
6	18th	......	172	211	387	154	215	308	434	466	282	188	160	116
7	18th	105	176	213	392	159	223	309	440	472	283	191	166	120
7	19th	109	185	217	415	168	237	332	474	510	304	202	174	124
8	19th	89	145	175	330	133	184	257	366	395	234	156	129	102
9	19th	57	95	101	188	76	106	149	204	216	123	93	67	61
9	20th	60	91	93	178	80	103	140	195	208	113	91	68	61
9	24th	56	83	96	147	......	94	134	176	187	101	83	61	59
10	June 5th	51	*	*	*	*	*	118	157	169	93	*	*	*

*These points were inaccessible. Note that the deflections given for Point 27 represent deflections from the position of the slab under Load 1. At all other points the deflections shown are from the initial position of the slab.

TABLE 12.—Test of Northwestern Glass Company Building.

Steel Stresses—Upper Side of Slab.

Stresses are given in thousands of pounds per square inch.

Load No.	Date, May, 1913.	Stresses at Gauge Line Number.										
		101	102	103	104	105	106	107	108	109	110	111
Zero	13th	0	0	0	0	0	0	0	0	0	0	0
1	13th	0.4	2.2	2.8	0.4	0.8	1.7	2.0	2.8	1.3	0.2	2.4
2	14th	1.5	5.8	7.1	2.0	2.7	9.7	9.3	11.6	6.4	2.0	12.0
2	15th	2.0	5.8	7.0	2.1	2.7	9.9	9.8	12.1	6.4	2.0	13.6
3	15th	2.8	7.9	9.7	2.8	2.4	14.8	15.8	18.7	10.1	2.8	20.2
3	16th	2.7	8.7	9.7	2.9	2.6	15.5	17.5	20.0	10.6	2.8	22.4
4	16th	3.0	11.2	11.2	4.5	3.7	13.8	15.5	17.8	9.8	2.1	19.6
4	17th	3.1	12.1	13.6	4.9	3.6	14.3	15.3	18.0	10.2	2.7	19.3
5	17th	3.4	16.3	18.3	7.4	5.2	10.9	10.6	13.0	9.4	2.1	14.2
5	18th	4.0	17.3	18.8	8.3	5.9	12.1	10.3	13.9	10.0	2.8	14.4
7	18th	4.2	18.1	19.3	8.5	5.8	11.4	7.6	11.9	9.8	2.8	14.4
7	19th	5.2	19.3	19.8	9.7	7.3	12.0	8.7	12.1	10.1	3.4	14.7
9	19th	3.6	5.8	6.8	4.4	5.6	8.2	7.7	8.1	5.1	0.9	11.6
9	20th	3.3	4.7	5.6	3.5	4.3	7.1	7.0	7.6	4.2	0.1	10.3

All stresses in this table are tensions.

TABLE 13.—Test of Northwestern Glass Company Building.

Steel Stresses—Under Side of Slab.

Stresses are given in thousands of pounds per square inch.

Load No.	Date, May, 1913.	Stresses at Gauge Line Number.										
		1	2	3	4	5	6	7	8	9	10	11
Zero	12th	0	0	0	0	0	0	0	0	0	0	0
1	13th	1.0	0.2	1.5	0.8	1.1	2.0	0.4	0.5	1.0	−0.4	1.5
2	14th	2.0	1.7	3.0	2.3	3.7	4.0	3.3	3.6	4.9	4.9	4.0
2	15th	1.8	1.7	2.7	1.2	2.7	4.6	2.8	3.4	4.7	5.2	4.0
3	15th	2.2	3.0	4.8	2.8	4.0	6.1	5.7	6.7	8.1	12.6	9.9
3	16th	2.5	3.4	4.6	2.8	4.3	6.4	6.5	7.5	8.2	15.0	12.1
4	16th	3.4	5.3	7.7	4.0	6.1	7.9	7.0	7.2	6.8	16.3	17.8
4	17th	3.9	5.8	8.4	4.8	6.9	8.8	7.5	7.1	7.5	17.0	19.0
5	17th	6.0	10.9	14.2	8.6	11.7	11.8	10.0	9.0	7.6	14.9	19.2
5	18th	7.1	12.2	15.7	10.3	14.2	13.5	10.9	8.8	6.9	15.1	20.5
7	18th	6.5	11.8	15.9	10.8	14.7	13.9	11.6	9.4	8.4	7.5	9.4
7	19th	7.1	12.5	17.1	11.9	16.6	15.3	12.7	10.7	8.2	6.7	9.0
9	19th	3.0	4.8	6.8	4.9	8.0	6.2	6.0	5.4	5.8	7.9	10.7
9	20th	3.1	4.4	5.9	4.4	7.1	6.1	5.5	4.3	4.5	7.9	9.6

The minus sign indicates compression. Where no sign is used the stress is a tension.

DISCUSSION

Mr. Godfrey.

EDWARD GODFREY, M. AM. SOC. C. E. (by letter).—The flat slab has repeatedly been brought before the Engineering Profession for consideration and adoption, on the plea that it is efficient and economical. Before adopting it, however, the Profession has a right to examine its credentials. The theory of the flat slab must pass muster; and tests on it must be consistent and rightly interpreted. Some things in reinforced concrete design have been adopted which have not met these conditions. They were adopted in ignorance and have been the cause of unnumbered wrecks. One of these is the stirrup or short shear member, another is the rodded column with no lateral reinforcement; but these will be rooted out in time. That they ever received the sanction of the Profession is a disgrace. It will not do to add to the number of such ill-considered standards by the adoption of any other untried and illogical method of design.

The theory of flat slabs, as given by most writers, would tend to place it in the innocuous class, for the great thickness of slab demanded would put it "out of the running"; but the commercial designers succeed in scaling down the bending moments to a point which independent practising engineers fear to attempt.

The flat slab theory premises a slab of indefinite extent, supported on an indefinite number of columns, the entire slab being uniformly loaded. The term, cantilever flat slab, so often used, conveys the idea that over the column heads the slab is a cantilever. The premises and the several assumptions are faulty. The slab is not of indefinite extent. It must end at the outer row of columns or a short distance beyond them, and sometimes there are only two or three bays. Sometimes there is a girder or wall at the outer edge and sometimes there is not. Manifestly, when the outer row of columns is reached, there must be—in order to satisfy the theory—something to take the place of the continuous slab that will supply its equivalent. A girder or wall on the line of the columns does not perform this office, as a girder cannot hold a slab in a fixed horizontal position as though it were a cantilever. A continuation of the slab beyond the column line cannot supply the condition necessary, unless it is uniformly loaded with just the proper balancing load. This uniform load, however, would of necessity include some live load, and there is no law compelling an owner to dispose the load on his floor in any given way.

The idea that the slab is a cantilever over the column means one of two things. Either the load on the slab must be properly balanced, or the columns must be subject to a bending moment due to live load to one side of the column line. To assume that the live loads are always properly balanced over the columns is to restrict the owner in the free use of his structure and to demand something unthinkable.

Mr. Godfrey.

To make the columns carry the cantilever load of the slab is to subject them to stresses for which they are never calculated.

On page 1346 the author says:

"The stress at 11 in the wall panel, C, was larger, and amounted to 20 500 lb., which probably was due to the cantilever action at the wall being less than that exerted by the usual column heads."

Here is a confession that the column heads—which means, of necessity, the columns—are depended on to take cantilever stresses from the floor.

It may be urged that the stiffness of the slab will make it capable of taking at least a part of the bending moment beyond the column line. It cannot possibly take all of it without either infinite stiffness or articulation in the column. Any theory is wrong that does not follow to an ultimate and adequate support or resistance the loads or strains assumed on part of a structure. A slab theory that ignores the effect of the slab on the columns, and leaves the reader to infer that these columns are centrally loaded, is wrong.

Another part of the theory that is wrong is the assumption that Poisson's ratio has any application to the steel reinforcement. The author's theory for the mushroom slab assumes that the steel reinforcement is equivalent to a continuous flat sheet of steel. This is contrary to fact and reason. A set of cross-rods does not in any sense fulfill the conditions that make it equivalent to a single sheet of steel.

It is freely admitted that when a sheet of steel is pulled in one direction it contracts in the other direction. As to whether this contraction is equivalent to a compressive force that would give the same amount of shortening is another question. If this is applied to two sets of transverse rods, an entirely different condition exists. The only medium between the sets of rods is the concrete. In a previous discussion* the author has stated that for the mushroom type Poisson's ratio is five-tenths. For the type that he delights to honor, then, here is what this statement means. A 1-in. square rod stressed to 16 000 lb. is pulled crosswise by the concrete 8 000 lb. for each inch of its length. The concrete might have an adhesion one one-hundredth of this amount. Where does the other ninety-nine-hundredths come from?

In asking the Profession to accept this mushroom theory, the author is asking us to consider the laws of matter suspended for this particular type, or else that there are special laws that apply to this special combination of materials. The theory of flat slabs is hard for any disinterested engineer to accept; let us look at the tests.

The author finds agreement between his theory and the result of tests. According to his statements, deflections and extensions in the

* *Proceedings*, Am. Soc. C. E., for August, 1913, p. 1368.

Mr. Godfrey.

steel would seem to bear out his theoretical deductions. This apparent agreement is explained by two things, one is tension in the concrete, and the other is large columns that are only partly loaded by the tests and, therefore, can lend some of their unused strength toward resisting cantilever action.

There can be no doubt that tension in the concrete plays a large part in resisting bending moments, and consequently carrying the loads. This is true of other forms of reinforced concrete, as well. In a simple beam or slab a very large part of the bending moment is resisted by tension in the concrete; but the accepted theory ignores this tensile strength entirely, and wisely so. This is the reason that deflection calculations for reinforced concrete beams, based on no tension in the concrete, are worthless.

In a flat slab, the tensile strength of the concrete has vastly more influence in supporting loads or resisting bending moments than in a simple beam, because of the fact that the tension is in all directions. Deflection is diminished and steel stresses are reduced by reason of this tensile strength of the concrete, and any theory that tries to explain these results through the supposed influence of Poisson's ratio is false. It cannot be gainsaid that, if a flat slab were cracked along certain lines, the steel stress and the deflection would be found to be very much more than they are in a whole slab, just exactly as the same would be true in any reinforced concrete beam or slab.

Tests on a flat slab, that are manifestly very greatly influenced by the tensile strength of the concrete are falsely interpreted, when the effect of that tensile strength is totally ignored, and, instead, Poisson's ratio (a thing that applies only to "any piece of material which is subjected to stress, and is of such shape that more than one of its dimensions is considerable,"* hence not to a rod) is brought in to explain the low stress in the steel.

If the commercial investigators frankly admitted that the tensile strength of the concrete is their mainstay, users of the flat slab would adopt it with their eyes open, if they had the temerity to use it.

The author has published† a test on a mushroom slab which purported to show the great strength of this style of construction and its agreement with theory. This slab, though only 18 ft. square, was supported on four comparatively enormous and very short reinforced concrete columns. It was loaded with balancing loads, that is, the first and second increments of the load were partly within and partly without the square enclosing the columns, which was 12 ft. each way.

Of course, such columns would have large influence in carrying cantilever loads, because of their 18 in. of width and their small direct load, but this is totally ignored in the author's theory, and all the

* As defined by Dr. Eddy in *Proceedings*, Am. Soc. C. E., for August, 1913, p. 1364.

† *Engineering News*, March 27th, 1913.

Mr. Godfrey.

strength is attributed to the virtue of the system. A building could not be commercially designed with columns about half as thick as their height and a safe load of less than 100 lb. per sq. in. This test is so far from representing anything in practice that it has not even a theoretic interest, especially when the theory absolutely ignores the greatest factors in supporting the loads.

Furthermore, balanced loads on this test slab tell nothing whatever of what the slab would do if the loading were confined within the square enclosing the columns.

The dishing effect of a flat slab is often referred to as explaining the strength exhibited in tests, and there is no doubt that for an interior loaded bay, with the surrounding bays idle, the dishing has a large influence. The writer has shown, by tests,* that though dishing in an interior panel greatly increases the strength of that panel, a row of loaded panels will not show this dishing, and will not have this large strength. The tests which have been made on flat slabs in buildings have been on one or more interior panels (including in some cases exterior panels supported on walls or girders) but never, so far as the writer can learn, on a complete row of panels across a building. Under such loading, the slab would tend to take a cylindrical shape instead of being dished, and the concrete and the steel would be under very much greater stress.

A flat slab on rows of columns is no better conditioned than the same slab supported on parallel lines of girders. No commercially designed flat slab will stand up under this criterion, that is, considering the slab supported on two lines of girders and figuring its bending moment as an ordinary slab subject to the common laws of Nature to which other than "flat slabs" are amenable. The writer has made these statements a number of times; they have never been controverted. This is the case of the flat slab in a nutshell. It stands or falls on the criterion just mentioned. In one notable case it fell with disastrous results.

The most serious aspect of this reliance on tension in concrete is in the case of rolling or jarring loads, as for a freight terminal or a viaduct. Repeated jarring of the load will in time crack the concrete. Then the steel will get its full load, which is several times that for which it is calculated, and trouble may be expected.

Mr. Eckles.

H. E. Eckles, M. Am. Soc. C. E. (by letter).—The assumption of a value for Poisson's ratio for steel and concrete in combination at four or five times its probable value, as shown by the published results of experiments with these materials in combination, and one-third larger than the sum of the ratios for the materials when not in

* *Proceedings*, Am. Soc. C. E., for August, 1913, p. 1358; and *Engineering and Contracting*, July 2d, 1913.

combination, and used by the author in one of his published works in the derivation of formulas similar to those presented in this paper, accounts for some of the discrepancies between former theories and conclusions based on facts. The variations in flat-slab design and the growing importance of this type of construction are so great that attempts to formulate conclusions should be made with care, and with a view to general applicability rather than in support of a particular type of construction. Mr. Eckles.

Much has been written regarding the action of flat plates under loads, and the writer believes that considerable of what has been put forth as a proper basis of design by advocates of reinforced concrete flat-slab construction is erroneous, and that the formulas proposed by the author in this paper give resultant stresses in the steel much less than would be developed under working conditions and proper test loading.

It should be noted that the results found by using the proposed formulas are compared with the observed stresses in the steel in the slab of Panel D when there was little load on the panels adjacent to it. In the adjacent Panel A the equivalent uniform load was about 260 lb. per sq. ft. of floor; in Panel C it was about 30 lb. per sq. ft.; and in the two other adjacent panels there was no load whatever. Our knowledge of this construction warrants us in believing that the effect of the surrounding floor in supporting the load of a single panel is quite large.

The writer recently conducted a test of a floor of the "mushroom" type of construction, having panels of the same dimensions and with a slab 7 in. thick, reinforced, however, with a larger percentage than usual of slab steel. The design load was 200 lb. and the superimposed test load was 400 lb. per sq. ft. of floor. The results of this test showed a remarkably large effect on the deflection at the middle of the panel, when parts of the adjacent panels were loaded with a full test load along two sides.

The adjoining panels were of the same dimensions and reinforcement as the one loaded. The panel tested was at the corner of the building, supported on two sides by wall beams, and corresponding to that of the corner panel shown in Fig. 1, having Columns 9, 10, 11, and 48 at the four corners.

In the first stage of the test, the panel only was loaded. Care was taken to leave clear passages so as to prevent any arch action.

The second stage consisted in loading areas 4 ft. 6 in. wide adjacent to two sides of the loaded panel, for the full length of the side of the panel, with the same uniform load as that on the panel. This placed a full test load practically over the full width of the belt of bars extending from column to column and parallel to the sides of the panel. After the placing of this additional load, the measured de-

Mr. Eckles. flection at the center of the panel was exactly twice that found when only the full area of the panel was loaded. The measurements were made very carefully by two direct methods, to avoid any error, and a period of 40 hours was allowed to elapse between the time of the loading of the panel and the loading of the adjacent strips of floor. It seems probable that the difference in the deflections would have been somewhat larger had the width of the adjacent loaded strips been greater.

This is in harmony with the results of tests given in papers* presented some months ago in which it was shown that loads are distributed laterally for a distance beyond the boundaries of the loaded area, in some cases amounting to more than one-half the span of the slab. Similar conclusions have been reached as a result of other tests. The method of arranging the reinforcement in this type of construction, shown in Fig. 1, with the belts of bars overlapping each other near the columns, would seem to indicate clearly that, for a single loaded panel, where the side belts are only partly loaded, the effective span of the diagonal belts would be decreased or largely modified. This effect would be enhanced still further by the fact that the side belts are of shorter spans and consequently subject to less deflection. This is precisely the effect of the "drop," used in the Larkin Building, and would seem to account for the approximate agreement of the results of the calculations by the author's use of his formula, with those of the Larkin test, where the load extended over several panels.

It has been pretty clearly established, by the numerous tests made, that the points of contraflexure are fairly close to that which the common theories of flexure would indicate, considering an elementary width of slab extending from column to column to act as a beam.

In the "mushroom" system this point is much nearer the column than in cases where the "drop" is used, and calculations made on this basis give results considerably in excess of those by the author's proposed formulas, based on partial test loads. The author has fallen into error through his disregard of the well-established laws of flexure, with the result that the conclusions in his paper are erroneous.

The advantages of the flat-slab type of construction are obvious. With the same quantity of material, the structure can be made approximately of the same strength as when the usual slab-and-beam type of construction is used, without excessive deflection, and with the added advantage, in many cases, of greater clearance, or a reduction of the cost by a less height of structure being required. With these facts in view, it is questionable practice to advocate a flat-slab type of construction designed under specifications inconsistent with what is recognized as good practice in the more common slab-and-beam type.

One of the disadvantages of the "mushroom" system without the

*At a meeting of the American Society for Testing Materials.

"drop" is the lessened strength in shear of the slab around the column head. At the beginning of the paper the author has advocated the use of small percentages of reinforcement, though his reasons for this are not apparent. The quantity of reinforcement often used in the "mushroom" system is one-fourth of 1%, and, considering the possible effect of the overlapping belts, with this percentage the neutral axis of the slab would be approximately three-tenths of the effective depth from the compression face at points near the column. In a slab of 8 in. effective depth, the neutral axis would be about 2½ in. from the face of the slab. Mr. Eckles.

The compression side of the slab is the only part of its cross-section which is effective in shear; the shear developed on this part of the slab around the top of the column is a measure of diagonal tension, and is of very high unit value. Where a "drop" is used, this unit shear is greatly reduced. With the same total quantity of reinforcement and a "drop" of half the thickness of the slab, the unit shear is reduced approximately one-fourth.

Sanford E. Thompson, M. Am. Soc. C. E. (by letter).—Mr. Eddy fails to bring out the following important conclusions from the tests on the Northwestern Glass Company's Building: Mr. Thompson.

1. The reinforcement of the slab at the column head is entirely inadequate for the design load, so that the tensile stresses in the steel and the compressive stresses in the concrete are excessive.
2. The gauge lines at the column head were not placed properly, so that the readings of the stresses do not represent the maximum stresses in the steel at the column head.
3. Tests made on single panels, or with the loading similar to those on the Northwestern Glass Company's Building, Loads 5 to 7, do not produce the largest stresses in the most important part of the structure, that is, at the column head.
4. The wall panels should be designed differently from the inside panels.

Stresses at the Column Head.—By the nature of the construction, the flat slab derives its strength from the rigidity of the column head. For this reason, it is absolutely necessary to design the slab so that the tensile and compressive stresses at the column head are within working limits. In analyzing the results from the tests, it is most important, therefore, to pay close attention to the stresses at the column head.

For the column head, the most unfavorable position of the loading is when all the panels around it are fully loaded. In the Northwestern Glass Company's Building, the only loading that caused the most un-

Mr. Thompson. favorable condition at the column head was when Load 3 was applied, and then the total load per panel was from 93 000 to 97 000 lb., though the panels were designed for a total live load of 102 400 lb. Even for this smaller load, however, the stresses in the steel due to the live load alone reached 22 000 lb. per sq. in. If to this is added the stress due to the dead load, the total stress for the design load is more than 50% greater than that usually allowed in reinforced concrete construction. The stress referred to is at Point 111 (Table 12). Stresses just as high undoubtedly would have been found elsewhere, if the other gauge lines had been properly placed over the edge of the head, as explained later. That this stress was not abnormal is indicated by the uniformity with which it increased under the load and then decreased when the load was removed.

Mr. Eddy attributes the high stress at Point 111 to the fact that:

"It was necessary to pry the ends of the rods upward forcibly and hold them in this position by blocks under their extremities, thus putting them under considerable initial bending stress."

As the gauge line, 111, was on a straight bar belonging to the diagonal band, it is difficult to see how lifting the steel in the bands, which were not attached to anything, puts any initial bending stresses on it. Even if initial stresses existed, they would not have affected the readings of the stresses, as extensometer measurements do not indicate the actual stresses in bars, but only the difference between the stresses at the initial reading and at the respective reading under load. Any initial stress, like stresses due to the dead load, is not included in this difference.

Mr. Eddy states, further:

"At 111, Column 36, on the edge of the loaded area, the abnormal value of 22 400 lb. was reached under Load 3. Load 5 shows no such large increase over Load 3 of observed stress in the slab rods of the column heads in Panel D as was to be expected by practically doubling the loading. This fact is apparently inexplicable."

This statement is absolutely misleading. Point 111 is in Panel C, and this panel received its largest loading with Load 3. With Load 5, Panel C had a load of only 39 lb. per sq. ft., as may be seen by referring to the detail of loading in Fig. 6. It is entirely obvious, therefore, that the stress at this point ought to be less with Load 5 than with Load 3.

Faulty Location of Gauge Lines.—The location of the gauge lines over the column head is unfortunate. As is evident from Fig. 3, all gauge lines, except 103, 107, and 111, are inside the column head. The readings, therefore, on these gauge lines cannot represent in any way the actual maximum stresses in the reinforcement of the column head.

The writer has in his possession tests made by one of his associates

Mr. Thompson.

on a similar construction, where readings were taken on gauge lines placed as shown by Fig. 20. The average results from this test gave at A a stress of 11 000 lb. per sq. in., and, at the same time, the stress at B was 24 000 lb. per sq. in. Similar results were obtained in the Worcester test, made under the direction of Mr. Brown. It is evident, therefore, that for gauge lines located like those used in the Northwestern Glass Company's Building, the results are much smaller than the maximum stresses.

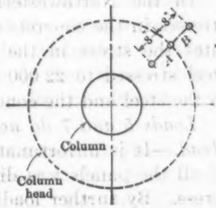

FIG. 20.

This rapid decrease in stresses explains why, in Table 12, all those except 107, 108, and 111, were very low. The low stress in 103, which was placed outside the column head, is evidently caused by the fact that the gauge line was too far from the column head, and the bar was one of the outside ones of the diagonal band.

That Mr. Eddy realizes that the stresses in the rods inside the cap are much smaller than those in the same rods just after they leave the cap, is evident from the following:

"Although that part of the length of the rod which is inside the cap has its elongation prevented by the mass of the cap, the part outside must have its elongation correspondingly increased to compensate for this loss, and, on the whole, be equal to that of the rods beside it."

In analyzing the results from the test, however, he accepts the stresses inside the cap as the maximum stresses.

Stresses in the Concrete at the Column Head.—Mr. Eddy does not mention the stresses in the concrete at the column head. In the first part of his paper, he analyzes the relation between the stresses in the steel and in the concrete, and the influence of the percentage of steel on the stresses in the concrete. This analysis would lead one to believe that the stresses in the concrete, in the tests described later, are within working limits. This, however, is not the case. The case of "under reinforcement," mentioned by Mr. Eddy, exists within the panel. At the column head, on the other hand, the percentage of steel in the bands alone is four times that of the steel in each band. Wherever the bars overlap, the percentage is still larger. In designs similar to those discussed in the paper, the percentage of steel at the column head is between 1.5 and 2. For these percentages of steel, the stress in the concrete, corresponding to the working stress in the steel, must exceed the allowable working stress. This is evident from Table 1.

It is of great importance for the safety of the structure to keep the compressive stresses at the column head, as well as the tensile stresses in the steel, within working limits. Therefore, if a large percentage of steel is used at the column head, as is always the case in this type of construction, compression steel is indispensable.

Mr. Thompson.

In the Northwestern Glass Company's Building, an idea of the stresses in the concrete due to live load alone may be had if one calculates the stress in the concrete corresponding to about 1.3% of the steel stressed to 22 000 lb. With the dead load included, the stresses in the steel and the concrete would be at least 25% greater.

Loads 5 and 7 do not Produce the Worst Condition at the Column Head.—It is unfortunate from a scientific standpoint that the loading of all the panels was discontinued as soon as Point 111 received high stress. By further loading of all the panels, their true strength would have been obtained, but with the loadings as used, the results are simply misleading.

Loads 4*a* and 5, for instance, show Panels B and D loaded and Panels A and C unloaded. With this load, the stresses at the column, instead of being taken by the steel in one-fourth of the circumference of the column head, that is, by the steel tributary to that panel, were taken by the steel in one-half of the circumference. The correctness of this statement may be seen by reference to Table 12 where it is shown that although Panels A and C were not loaded, Points 103, 107, and 111, showed considerable stress. From this it is evident that the loading of Panels D and B was carried at the column head, not only by the steel belonging to Panels D and B, but also by the steel in Panels A and C. The stresses at the column head, therefore, with this kind of loading, are much smaller than would have been the case had all spans carried loads of equal intensity.

Other Tests on Mushroom Floors.—The student of this type of construction is struck by the paucity of reliable test data on the particular type of construction discussed by Mr. Eddy. Many tests are constantly referred to in printed literature, but on closer examination one is surprised to find that they bring out the stresses everywhere except in the most vulnerable part of the construction.

The test to destruction carried on by C. A. P. Turner, M. Am. Soc. C. E., or on his behalf, described in the engineering papers and more fully in Mr. Eddy's book, might have elucidated many mooted questions. The idea of a test panel with projections, if loaded properly, was a very good one, because the load on the projections could have been arranged so that it would have an effect almost similar to loads on the adjoining panels. The idea, however, was not carried out, and the slab was broken by loads placed in the center panel with comparatively little load on the projections. The question of the stresses in the most important part, that is, at the column head, therefore, was neglected, with the consequence that the results of the test are simply misleading to any one not well versed in the subject.

The test of the St. Paul Bread Company's Building is another instance in which everything was tested except that which is ordinarily the weakest part of the construction.

Formulas.—Mr. Eddy's formulas do not agree with the results of the tests in the most important place, *viz.*, at the column head. Although he calls them "theoretical formulas", the theoretical foundation is nullified by the many assumptions which do not agree with the actual conditions, so that the final results can lay no claim to being theoretically correct. Mr. Thompson.

One of the assumptions for which no clear reason is given is that for the value of Poisson's ratio, K. As explained in Mr. Eddy's book, that ratio, for concrete alone, varies from 0.1 to 0.2. For steel alone this ratio is about 0.3. For a flat-slab construction, however, Mr. Eddy considers that neither of these two values is large enough, and he accepts a value of K equal to 0.5. To support this value, he has given a beautiful formula, but does not explain how two materials acting together can change their nature entirely, and how the top portion of a concrete slab, by merely being provided with steel at the bottom, can take on a lateral expansion equal to one-half the compressive deformation under stress. The writer does not see how this statement can be accepted by any one acquainted with the nature of the material, and yet on this assumption hangs the most vital element in the results.

The stresses for the steel in the diagonal and rectangular bands are calculated from formulas derived without regard to the size of the column head, and, according to Mr. Eddy, would apply to a construction with a column head equal to a sharp point and one of any size whatever. This assumption, on the face of it, is erroneous. The stresses in concrete at the column head are simply neglected.

In determining the stresses in steel over the column head, Mr. Eddy states: "The stresses in the middle rods of each belt, consequently, are increased abnormally for this reason just as it leaves the cap."

This statement, however, does not prevent him from continuing:

"Instead of attempting to determine this increase by some intricate investigation, it will be simply assumed that the stress at this point in the middle rod does not exceed that in the outside rod of the belt at a point opposite the center of the cap."

The available tests prove the correctness of the first statement. They show, however, that the stresses in the outer bars are much smaller than those in the middle bars. Mr. Eddy's assumption, therefore, is without foundation, and, as a consequence, the results from the formulas do not agree with the tests. In the Northwestern Glass Company's Building, by using the actual live load on the panel, there is obtained by his formula a stress, $f_s = 15\,400$ lb. per sq. in., while the actual stress due to live load is at least 22 000 lb. per. sq. in.

In discussing the tests, Mr. Eddy uses, in his formulas for the total load in the panel, a much larger load than was actually on the panel. As a justification for this, he states that portions of the panels near

Mr. Thompson.

the column heads were not loaded. He overlooks the fact, however, that there were two center aisles about 18 in. wide running across the center of the panel and thus taking away the load from the place where it would be most effective in producing bending moment. The use of the larger load is still more unjustifiable in calculating the stresses over the column head, because the stresses there are more affected by the amount of load in the panel than by their position. It appears that the same load uniformly distributed would not have caused larger stresses than was caused by the load placed as shown in Fig. 5.

In discussing the Larkin Building, Mr. Eddy makes the following statement:

"It would require a load of more than 1 200 lb. per sq. ft. on such a mushroom slab to produce a maximum unit stress on the slab rods as great as was caused in the Larkin slab by the test load of 738 lb. per sq. ft."

This, of course, is calculated according to his formula, which, as has been shown, does not agree within 45% with the actual results. This claim is only another expression of Mr. Eddy's faith in the miraculous properties of this particular type of reinforcement; but, to engineers familiar with the design of reinforced concrete, it seems at least open to discussion.

As a conclusion, it appears, from a careful study of the tests, that a design similar to that of the Northwestern Glass Company's Building does not have the proper factor of safety required in reinforced concrete construction; that the stresses in the steel, as well as in the concrete, as is evident from Table 12, exceed those used in conservative designs; and that the formulas given by Mr. Eddy do not appear to agree with the results with the accuracy claimed by him; in fact, as stated before, the difference between the results from the formulas and the actual stresses reaches the large amount of at least 45 per cent.

Mr. Mensch.

L. J. Mensch, M. Am. Soc. C. E. (by letter).—This paper appears to the writer to be a defense of, and a propaganda for, the so-called mushroom type of flat floor construction.

The author seems to be quite satisfied that mushroom floors with a notoriously deficient quantity of steel reinforcement are safe and proper constructions. This is a debatable question, with reference to the factor of safety required. In some constructions there is a factor of safety of a little more than 1, as in foundations, retaining walls, dams, bridge abutments, etc., in fact, where the carrying capacity and cohesion of the ground is to be considered. In other cases a factor of safety of hardly 2 is often, knowingly or unknowingly, adopted, as in columns, whether of steel, brick, or concrete, especially in outside columns and those which are eccentrically loaded, also in roof trusses, common brick walls in buildings, etc.

The numerous cases where such low factors of safety are found may blind an engineer and make him believe that he is as well justified in adopting a similar factor of safety in flat slab construction. On the other hand, engineers are accustomed to allow a factor of safety of 3 in steel-girder constructions, and the general public demands a factor of safety of at least 4 in reinforced concrete construction, on account of its novelty and its greater uncertainty in erection. Mr. Mensch.

The writer claims that a large percentage of the examples cited by the author have a factor of safety of a trifle more or less than 2, and that his formulas simply accommodate the extensometer tests of these slabs. Extensometer tests do not give any indication of the real strength of a slab, which is reinforced by light bars and to such low amounts as ¼ per cent. The extensometer gives the elongation in a length of generally 8 in. of steel bars embedded in concrete. Where the surrounding concrete is not cracked in the 8 in., the extension obtained by the extensometer, in inches per inch of length, multiplied by the modulus of elasticity of the steel, generally taken at 30 000 000, will give the steel fiber stress, but no correct idea of the interior resisting moment, because the tensile strength of the concrete is disregarded. Assume, however, that there is a crack in the concrete between the contact points of the extensometer, then the stress is transferred by shear and by the bond of concrete and steel to the steel bar in the neighborhood of the crack, and the bar is stressed considerably higher at the crack than at points farther from it.

For an illustration, assume that the reinforcement consists of ⅜-in. round bars, 6 in. from center to center, that the crack in the concrete is 1 in. deep, and that the tensile strength of the concrete is 200 lb. per sq. in. The surface of a ⅜-in. bar per lin. in. is 1.18 sq. in., and its cross-section is 0.11 sq. in. The original strength of a concrete section, 6 in. wide and 1 in. deep, is $6 \times 200 = 1\,200$ lb., and, assuming a bond stress of 400 lb. per sq. in., or 472 lb. per lin. in. of ⅜-in. bar, it will require a length of 3 in. on each side of the crack to transmit the original strength of the cracked portion of the concrete into the bar. This additional stress in the steel fibers amounts to $1\,200 \div 0.11 = 10\,900$ lb. per sq. in., and, for determining the elongation of the bar we have to assume that it acts only on an average of 3 in., though the extensometer measures on 8 in.; hence the stresses obtained by extensometer readings are in this particular case only three-eighths of the actual stresses. In other words, when one tries to judge, from extensometer readings, the distribution of the exterior bending moments, or their equivalent, the interior resisting moments, one is likely to under-estimate the value of the exterior bending moments 50% and more, due to the neglect of the tensile stresses of the concrete.

The fact that extensometer readings do not permit any correct conclusions as to the interior resisting moment, or the factor of safety of

Mr. Mensch.

TABLE 14.—Log Sheets of Gravel Concrete Beams, 13 Weeks Old.

Applied load, in pounds.	$\frac{M}{bd^2}$, in pounds per square inch.	Deformation, in Millionths of an Inch per Inch.		K, percentage.	Deflection, in inches.
		Upper fiber.	Steel fiber.		
Beam 337; ½% Reinforcement.					
0	24.85	17	21	44.6	0.010
1 000	54.85	48	44	52.0	0.020
2 000	84.85	66	63	51.3	0.030
3 000	114.85	112	115	49.4	0.050
3 500	129.85	138	156	46.9	0.060
4 000	144.85	172	236	42.2	0.080
4 500	159.85	237	472	33.5	0.130
5 000	174.85	282	639	30.6	0.170
5 500	189.85	319	815	28.1	0.215
6 000	204.85	359	993	26.5	0.270
6 500	219.85	398	1 162	25.5	0.310
6 740	227.05	....	1 241	......	0.340
5 600	192.85	491	1 154	29.9	0.340
Beam 338; ½% Reinforcement.					
0	24.64	21	19	52.5	0.010
1 000	54.64	51	43	54.1	0.025
2 000	84.64	75	63	54.3	0.035
3 000	114.64	115	109	51.4	0.055
3 500	129.64	142	140	50.4	0.060
4 000	144.64	182	217	45.6	0.085
4 500	159.64	235	373	38.6	0.120
5 000	174.64	286	566	33.5	0.170
5 500	189.64	337	797	29.7	0.225
......	....	....	....	......	....
5 000	174.64	411	1 684	19.6	0.330
Beam 348; ¾% Reinforcement.					
0	25.26	23	15	60.4	0.010
1 000	55.26	53	38	58.0	0.025
2 000	85.26	77	56	57.8	0.035
3 000	115.26	113	85	57.0	0.050
3 500	130.26	126	103	55.1	0.055
4 000	145.26	146	127	53.4	0.060
4 500	160.26	167	168	49.8	0.070
5 000	175.26	209	253	45.3	0.100
5 500	190.26	257	362	41.5	0.120
6 000	205.26	293	472	38.3	0.150
7 000	235.26	357	656	35.2	0.210
8 000	265.26	421	865	32.8	0.270
9 000	295.26	481	1 072	31.0	0.330
9 370	306.36	....	1 154	......	0.355
9 620	313.86	....	1 338	......	0.390
8 000	265.26	534	1 726	23.7	0.395
Beam 349; ¾% Reinforcement.					
0	25.47	13	15	47.2	0.010
1 000	55.47	42	39	51.9	0.020
2 000	85.47	64	60	51.7	0.040
3 000	115.47	93	84	52.5	0.050
4 000	145.47	117	111	51.4	0.065
4 500	160.47	145	149	49.3	0.080
5 000	175.47	171	207	45.2	0.100
5 500	190.47	217	321	40.3	0.120
6 000	205.47	257	456	36.0	0.150
7 000	235.47	321	706	31.3	0.220
8 000	265.47	380	913	29.4	0.280
9 000	295.47	433	1 087	28.5	0.330
9 500	310.47	464	1 197	27.9	0.360
10 000	325.47	489	1 294	27.4	0.390
10 500	340.47	516	1 378	27.2	0.415
11 000	355.47	551	1 467	27.3	0.440
11 050	356.97	....	1 516	......	0.465
9 300	304.47	548	1 981	21.7	0.475
Beam 350; ¾% Reinforcement.					
0	25.06	19	21	47.5	0.005
1 000	55.06	43	46	48.6	0.020
2 000	85.06	70	68	49.6	0.030
3 000	115.06	102	96	51.5	0.040
4 000	145.06	133	130	50.6	0.050
4 500	160.06	160	169	48.7	0.060
5 000	175.06	198	248	44.4	0.080
5 500	190.06	244	390	38.5	0.115
6 000	205.06	289	533	35.2	0.150
7 000	235.06	360	754	32.3	0.210
8 000	265.06	418	945	30.7	0.265
8 800	289.06	....	1 250	......	0.345
7 500	250.06	498	1 597	23.8	0.355
Beam 360; 1% Reinforcement.					
0	25.26	23	17	57.5	0.010
1 000	55.26	56	43	56.5	0.015
2 000	85.26	84	65	56.5	0.025
3 000	115.26	107	87	55.1	0.035
4 000	145.26	135	111	54.9	0.045
4 500	160.26	158	130	54.9	0.050
5 000	175.26	181	162	52.8	0.060
5 500	190.26	206	222	48.2	0.080
6 000	205.26	246	330	42.7	0.100
7 000	235.26	309	508	37.8	0.150
8 000	265.26	358	643	35.8	0.190
9 000	295.26	409	786	34.2	0.230
10 000	325.26	455	938	32.6	0.275
11 000	355.26	508	1 068	32.2	0.320
12 000	385.26	556	1 200	31.7	0.360
12 850	410.76	....	1 362	......	0.400
10 700	346.26	626	1 921	24.6	0.430

Mr. Mensch.

the slab, is indisputably proved by tests made by Richard L. Humphrey, M. Am. Soc. C. E.* In that investigation there were 336 beams, 13 ft. long, 8 by 11 in. in cross-section, and they were tested on a 12-ft. span by two equal loads at the one-third points. The reinforcement throughout consisted of $\frac{1}{2}$-in. bars, having a yield point of approximately 40 000 lb. The number of bars varied from two to eight, corresponding to a percentage of $\frac{1}{2}$ to 2. The mixture of the concrete was 1:2:4 by volume, and the tests were made when the beams were 4, 13, 26, and 52 weeks old. In every beam tested there were careful readings of the deformations, both of the top fibers and of the steel fibers, for stages of loadings of from 500 to 1 000 lb. up to the ultimate load.

In Table 14, taken from Technologic Paper No. 2, it will be noticed that Beam No. 337 failed at a total load of 6 740 lb. At a load of 3 500 lb., which is more than one-half of the ultimate load, the extensometer reading of the steel fiber was $\frac{156}{1\,000\,000}$ in. per in.; or, multiplying this extension by 30 000 000, we obtain a stress of only 4 680 lb. It is well to keep in mind that the extensometer reading gives such a low stress when the factor of safety is less than 2. Similarily, we find that at a load of 4 500 lb., which is two-thirds of the ultimate load, the stress in the steel fibers is 14 160 lb. At 5 000 lb., which is 75% of the ultimate load, the steel fiber stress is only 19 000 lb. per sq. in. Similar conditions may be found in every beam tested, even in those with 2% of reinforcement. One may reasonably expect that the discrepancies will be still greater in flat slab construction, where the percentage of reinforcement is considerably lower than in Beam No. 337, being mostly only $\frac{1}{4}$ to $\frac{1}{3}$ of 1 per cent. One can even imagine the case where, with an insufficient quantity of reinforcement, the slab will break at practically the same load as a non-reinforced slab, where, just before breaking the extensometer would not show a larger extension that 0.0001 in. per in., corresponding to a stress at failure of only 3 000 lb. per sq. in.

In studying the tables it is found that at certain stages a very small increase of the load causes a great change in the extensometer readings, which explains the great discrepancies found by Mr. Eddy in the stresses of adjoining bars or of symmetrical points. The unreliability of ex ensometer readings for the determination of the interior resisting moment of reinforced concrete beams was discussed by the writer before the Western Society of Engineers in the spring of 1904.†

There is yet another simple indication that the flat slab constructions cited by Mr. Eddy have not the conventional factor of safety, and that is the appearance of cracks at the most dangerous sections, where one would expect to find them at loadings of from one-third to one-half of the ultimate load. From tests of beams reinforced by $\frac{1}{4}$%, it is

* Technologic Paper No. 2, U. S. Bureau of Standards.

† The *Journal* of the Western Society of Engineers, for 1904, contains a number of tables and diagrams.

Mr. Mensch. known that cracks appear at a load which is about three-fourths of the ultimate; and, in slabs reinforced by $\frac{1}{2}\%$, at about one-half of the ultimate load.

Mr. Eddy writes:

"Indeed, slab action in general may be described partly as the attempted mechanical superposition of one set of parallel depressions and elevations on another set of similar corrugations at right angles to them. Such sets mutually support each other and give rise to slab action. * * *"

This sounds mysterious, and will not enlighten any one; it only shows that the author does not care to express in a scientific way the real slab action, as has been done, for example, in a masterful way, by the Italian engineer, Danusso,* and by others.

There is no mystery about slab action. It is simply a combination of continuous beams acting in at least two directions, which beams are connected with each other and with the columns, and the deflection in any point is common to at least two beams acting at right angles to each other.

On account of the large sizes of the columns and column heads, and the depression of the floors at the columns, the beams directly connecting the columns, and corresponding to the sides of the squares, have considerably greater stiffness than those parallel to them and nearer the center of the slab, and these beams have a comparatively still greater stiffness than any diagonal beams, with the result that the mechanical action of the whole slab must be nearly identical with the action of a flat slab of a smaller span than the distance from center to center of columns, supported by four wide girders of a depth somewhat greater than the thickness of the slab.

If we omit to take into account the presence of the large columns and column heads, we obtain the case considered by Winkler and Grashof, who assumed the supports of the slab to consist of ideal points, and who found that the bending moment in the center of the span is $\frac{W L}{48}$ and over the columns and side of the square is $\frac{W L}{24}$, when W is the total load on the panel, L is the distance from center to center of columns, and Poisson's ratio is 0.1. The bending moments change only very slightly if other values of Poisson's ratio are assumed. For steel, it is generally found at 0.3, and the moments become $\frac{W L}{26.4}$ and $\frac{W L}{52.8}$, respectively. Returning again to the flat slab as a combination of four wide beams supporting a smaller flat slab, it is only within reason to assume that the wide girders have to carry the same loads as ordinary beams in the case of a flat slab supported by narrow and deep beams.

* "Il Cemento," 1912.

It is here that one finds a riotous license of figuring by most of the advocates of flat slab constructions. According to most building ordinances and accepted good practice, the bending moments in beams supporting flat slabs must be taken both over the supports and in the center of the beams at not less than $\frac{W}{2} \times \frac{L}{12} \times \frac{3}{2}$ (the latter factor on account of the triangular application of the load over the beam) $= \frac{W L}{16}$, although theory shows that the bending moment in the center of the span is only $\frac{W L}{32}$. Mr. Mensch.

Now, the advocates of the mushroom system are not satisfied to figure the bending moment in the center of the side beams as the theoretical moment, which is only one-half of the moment any engineer is allowed to figure when he adopts common girder construction; they go still further, and claim the right (according to Mr. Eddy's Equation 34) to figure this moment as $\frac{W L}{175}$, which is one-tenth of the moment required by the building ordinances. There we hear again that they must be right or their constructions would fail at the first application of the design load, and that they are right because the extensometer readings bear them out. The writer has shown how erroneous are the results obtained by extensometer readings; it remains yet to prove that the real bending moments in the center of the side beams are less than are required by the building ordinances, otherwise their structures actually would not stand up.

In flat slabs supported by continuous girders of slightly greater stiffness than the slab, the Italian engineer, Danusso, has shown that the load of the slab is distributed nearly uniformly over the girders, instead of with the triangular distribution found for very stiff beams; hence the bending moments for a beam supporting two adjoining panels are theoretically $\frac{W L}{24}$ and $\frac{W L}{48}$, at the supports and the center, respectively. Engineers need not be surprised that the bending moments for the stiffer side beams are just as large as those given by Grashof for the whole width of the slab. This case happens very often in ordinary girder and joist construction. Assume, for example, a flat slab supported by parallel walls, 20 ft. from center to center. If this distance is spanned by a simple slab, the bending moment is $\frac{W L}{8}$. As a rule, it is cheaper to provide girders of 20 ft. span and, say, 10 ft. from center to center, which girders must be figured for a moment of $\frac{W L}{8}$, and the slabs between the girders must be figured for another bending moment. Hence the sum of all bending moments

Mr. Mensch. is larger than the original moment $\frac{WL}{8}$, yet the cost of the construction is generally less.

In continuous beams which are of larger moment of inertia near the supports than in the center, such as is the case with the large columns and column heads used, it is known, also, that the moments at the supports become larger and those in the center of the span become smaller; taking also into consideration the fact that the span of the side beams may be safely diminished by one-half of the diameter of the columns, the moments of $\frac{WL}{24}$ and $\frac{WL}{48}$ become in most cases cited by Mr. Eddy, where no depression in the floor is used, $\frac{WL}{24.4}$ and $\frac{WL}{62}$; and, where a depression is used, they become $\frac{WL}{22.2}$ and $\frac{WL}{92}$. Hence, in the most favorable case, giving the mushroom system all the advantages of continuous action, such as is not adopted by any responsible designer in any other class of work, neglecting at the same time all possible settlements, Mr. Eddy advises the use of a bending moment of one-half of what it can possibly be, which readily explains the low factor of safety.

The advocates of the mushroom system may claim that the diagonal beams help to support a great portion of the load which the side beams are assumed to carry. Mr. Eddy, however, admits that the stresses in the center of the diagonal beams are generally found to be one-half as great as those in the side beams; this, together with the fact that the span of the diagonal beams is 1.41 times that of the side beams, permits the conclusion that a diagonal beam supports only one-quarter of the load of a side beam, and, in fact, simply transmits the loads from the interior of the panel to the side beams.

Mr. Eddy advises calculating the diagonal beams in the center for a moment of about $\frac{WL}{250}$, which does not seem to be sufficient, even for the simple transmission of the loads to the side beams. Granting that the width of the side beams may be assumed to be four-tenths of the span from center to center of columns, the width of the interior slab is six-tenths of L and the bending moment per linear foot of the interior slab, according to most building ordinances, must be figured as $\frac{w\left(\frac{6}{10}L\right)^2}{24} = \frac{wL^2}{66.67}$, while $\frac{WL}{250}$ per diagonal beam corresponds to $\frac{wL^2}{150}$ per lin. ft. of the interior slab.

It is remarkable that Mr. Eddy overlooked the fact that the distance between the points of inflection on the sides of the squares of several of the tests cited by him were found to be from $0.5L$ to $0.55L$. According to his own statements, the bending moment must be larger in the center of the sides than $\frac{w.(0.5\,L)^2}{8}$ and $\frac{w.(0.55\,L)^2}{8}$, or $\frac{w.L^2}{32}$ and $\frac{w.L^2}{27}$, respectively, while his moment of $\frac{W.L^2}{175}$ corresponds to $\frac{w.L^2}{70}$. **Mr. Mensch.**

The advocates of many flat slab systems declare that the common beam theory does not apply to flat slabs. The writer has never seen a similar statement in the works of St. Venant, Winkler, Grashof, Föppl, or Müller-Breslau, who, in much more difficult problems than this, applied only the common beam theory, and in particular, the case of flat slabs was solved by Winkler and Grashof by that theory. There is yet one defense open to the advocates of the mushroom system, and that is the mystic influence of Poisson's ratio, but Mr. Eddy himself states that, on account of bending moments of opposite signs acting at right angles at the center of the side beams, the stresses there are larger than in a common beam; hence he can use it only for a defense of the low moments he assumes over the columns, and he claims that Poisson's ratio must be assumed as one-half. This, however, is a mechanical impossibility, clearly excluded by Grashof, as a simple change in shape without any change in the distance of the molecules is identical with a Poisson's ratio of one-half. Over the columns we have the most unfavorable combination of maximum bending moment and maximum shear acting together in the same section, and Grashof advised, even in the case of steel, reducing the stresses over the columns by 20 per cent. Besides, near the ultimate load, the concrete is cracked in every direction around the columns, and the steel bars will hardly be benefited by stresses acting in two directions, although the concrete in compression may be helped considerably. The percentage of reinforcement being low, an increase of the compressive stress of the concrete of 50% will benefit the internal resisting moment less than 5 per cent.

Mr. Eddy considers only the case of all panels loaded, which is a more favorable one for the center of all beams than that of a single panel, or two panels loaded, and extensometer tests clearly bear this out. In the discussion of a former paper on this subject,* the writer has shown that in the latter case the slab and the supporting columns and the columns above the slab form an arch construction in which the columns are very highly strained by the negative moments at the supports, and that the negative moments in the adjoining panels are very small when the columns have a larger moment of inertia than

* *Proceedings*, Am. Soc. C. E., for August, 1913.

Mr. Mensch. the slab. These conclusions were proved indisputably by the tests recently made by Mr. W. A. Slater, of the University of Illinois, on a new factory building of the Shredded Wheat Company, at Niagara Falls, N. Y. Mr. Slater presented the results of the test in a paper before the meeting of the American Concrete Institute in Chicago, in February, 1914. Although the floors were tested to only $1\frac{1}{2}$ times the total dead and live load, numerous fine cracks appeared, and the extensometer indicated steel stresses of 15 000 lb. per sq. in. in the slab and additional compressive stresses in the columns, due to the loading of panels on one side of the columns, of 400 lb. per sq. in.

Mr. Eddy declares that the drops in the floors are unsightly, bulky, and unnecessary, and prefers the mushroom type, where the depression of the floor is omitted and replaced by a magic steel ring in the neutral axis of the slab. That this advocates very poor construction, not countenanced by any reputable engineer, may be seen from the detail of the slab of the Northwestern Glass Company's Building, cited by Mr. Eddy. The column caps scale 36 in. in diameter, the dead weight of the floor is 120 lb. per sq. ft., the live load is 400 lb. per sq. ft., and the total load of a panel supported by one column is $16 \times 17 \times 520 =$ 141 440 lb. The floor slab is 8 in. thick, and the sectional area of the slab in shear around the cap equals $36 \times 3.14 \times 8 = 904$ sq. in., or a shear of $141\,440 \div 904 = 155$ lb. per sq. in. of the total section, or of about 220 lb. per sq. in. for a section of the depth of jd. Most building ordinances allow only a shear of 125 lb. per sq. in. on the section of a depth of jd, and prescribe the use of bent-up bars and stirrups at the section of great shear; on the other hand, the advocates of the mushroom system and their imitators take another license to step over ordinances and sober reasoning, allow a high shear, and no scientific reinforcement for shear whatsoever.

When called to task for allowing such a high shearing stress, these experts defend it by saying that only "punching shear" is acting. The writer does not know that "punching shear" is a term common in engineering, but, if it means anything, it must mean pure shear, like that in a true punching operation, when practically no bending stresses are acting on the material which is being punched. We have shown before that around the columns the maximum moment and the maximum shear are acting together, and that the stresses must be reduced and not increased at that section.

A great many thoughtless and entirely erroneous statements in reference to flat slabs may be found in most articles published on this subject in current engineering literature, and an example is Table 15.

From Table 15 it would appear that a flat slab, of 20 ft. span and a thickness of 8 in., contains per panel 784 lb. of reinforcing steel, if designed according to Grashof for a total dead and live load of 300

lb. per sq. ft. The writer has shown that Grashof's moments for the center and the sides of the square are $\frac{W.L}{48}$ and $\frac{W.L}{24}$, $W = 400 \times 300 = 120\,000$ lb., $L = 20$ ft., and $\frac{W.L}{48} = 50\,000$ ft-lb. and $\frac{W.L}{24} = 100\,000$ ft-lb. for the entire width of the panel, or per lin. ft., 2 500 and 5 000 ft-lb., respectively. Mr. Mensch.

TABLE 15.

No.	Method.	Thickness of slab, in inches.	Steel per panel, in pounds.
1	Cantilever	8	2 189
2	Turneaure and Maurer	12	1 931
3	Grashof	8	784
4	Mensch	8	2 120
5	Turner	8	549
6	McMillan	8	1 084
7	Brayton	8½	1 900

This table is from a discussion by A. W. Buel, M. Am. Soc. C. E., in *Proceedings*, Am. Soc. C. E., for August, 1913.

Adopting an allowable fiber stress for the reinforcing steel of 13 000 lb. (although it is too high for mild steel where a factor of safety of 4 is desired), and assuming the distance of center of reinforcing to the compression face as 7 in. (which is a trifle too small for the center of the slab and considerably too large for the sides of the square), we find that the sectional area of the reinforcement for a bending moment of 2 500 ft-lb. per lin. ft. in the center of the slab is 0.33 sq. in. and double the area is required at the sides of the square.

Where the negative reinforcement is obtained by bending all bars up at the quarter point, and continuing the rods to the quarter point in the adjacent panels, the weight of the steel reinforcement per linear foot of the panel in one direction is $0.33 \times 3.4 \times (20 + 2 \times 5) = 34$ lb., and for the entire panel in both directions $34 \times 20 \times 2 = 1\,360$ lb. It is very poor practice to bend up all bars; this would not be permissible in any girder construction on account of reversal of moments which may occur by settlements and unfavorable loadings in other panels. Assuming that one-third of the bars are straight and extend only 1 ft. beyond the center line at each end, and that, therefore, extra rods are required on top over the sides of the square, we find that at least 340 lb. of additional reinforcement must be placed in each panel, or a total of 1 700 lb.

Hence, according to Grashof, it requires in continuous panels at least three times as much steel as used in the Turner system, provided that the statement in Table 15 in regard to the Turner system is correct. Inasmuch as we have to design for the case of only two panels

Mr. Mensch. loaded side by side, it requires a great deal more reinforcing than the 1 700 lb., not counting the additional reinforcing which is absolutely necessary in the columns to overcome the bending stresses produced by one-sided loadings.

The writer begs not to be misunderstood as being an opponent of flat-slab constructions. He thinks he was the first to use flat slabs on a large scale, for he built two slabs, 100 ft. square, at the new plant of the Proctor and Gamble Company's soap works at Armourdale, Kans., in 1903, and has built many others since. He protests, however, when engineers and contractors (who, as a rule, have all to gain and nothing to lose), represent by versatile agents that flat floor constructions, of designs as advocated by Mr. Eddy, are as good as, or better than, girder constructions, which actually have a factor of safety of 4 or more, if they are designed according to the Chicago Building ordinance, for example, and if they are constructed right. No wonder that they can show to unsuspecting architects and owners a great saving over all other designs, and, being able to mention a great number of examples of buildings which did not fall down (having a factor of safety of about 2), are believed to be by the owners and architects very wizards in the art of reinforced concrete construction. It is true, however, that flat slabs show a slight saving (even if they are designed for a factor of safety of 4) over girder construction of the same factor of safety. The writer has not the reputation of wasting any material in the structures which he designs and builds; being a contractor, it would simply diminish his profits, yet he can mention a number of examples which were actually loaded with about four times the total dead and live load without breaking down.

In December, 1902, a girder in the Salvation Army Building, in Cleveland, of 24 ft. span and designed for a load of 100 lb. per sq. ft., was tested with 100 000 lb., which is about $3\frac{1}{2}$ times the total dead and live load. The girder showed some cracks, but no indication of a near failure, and the permanent deflection was $\frac{1}{4}$ in.

In the spring of 1905 the writer built a furniture storage house for the Brown Transfer and Storage Company, at St. Joseph, Mo. The structure was designed for a live load of 100 lb. per sq. ft. In the fall of 1905, when the writer started another building for the same company, he found that the building was rented to the Parker Grocery Company (which lost its own building by fire) and was loaded all over to the ceiling with canned goods weighing at least 600 lb. per sq. ft., so that both floors and columns withstood nearly four times the total dead and live loads.

In 1905 the writer designed and constructed a building for the Seng Company, Dayton Street, Chicago. The building was designed, according to the new Chicago ordinance for reinforced concrete, for a live load of 100 lb. per sq. ft. On account of some failures of con-

crete buildings all over the country at that time the owner refused payment on the building as the work progressed until such time as the writer could prove that the building had a factor of safety of 4. In the presence of a great number of engineers and architects, the writer tested one panel, only one month old, in December, 1905, with 600 lb. per sq. ft. The girders and slabs deflected and cracked considerably, but did not fail under the load, which remained there 24 hours. The permanent deflection was only ⅛ in. Mr. Mensch.

These examples should suffice to show that girder and slab constructions, correctly designed and built, have a factor of safety of at least 4, and there is no question that flat slab constructions can be made as safe as any others, but the writer claims that flat slabs designed according to Mr. Eddy's formula have only a factor of safety of 2, under the most favorable circumstances.

W. K. HATT, M. AM. SOC. C. E. (by letter).—The recent extension of testing operations to completed structures has shown, what many have known, that reinforced concrete structures carry their working loads in service by the help of tensile stresses in the concrete. The stresses in the steel may not be more than one-half or one-third of those which would be calculated by the ordinary formulas, which neglect these tensile stresses. Mr. Hatt.

For the purpose of standardizing design, and as a matter of safety, the conventions of the building codes properly omit these tensile stresses. They cannot be ignored, however, when the complete action of the structure is to be accounted for under working loads.

The variability of the quality of concrete not only makes it desirable to omit these tensile stresses in calculations for design, but it also renders the steel stresses, which are measured in tests of buildings, a very uncertain factor; that is to say, the measured stresses in the steel depend very largely on the quality of the concrete. Good concrete may relieve the steel of the greater part of its expected moment of resistance.

It follows, therefore, that, in judging the mechanical action of any type of structure, more attention should be paid to the measured compressive stresses in the concrete than to the measured stresses in the steel.

GEORGE S. BINCKLEY, M. AM. SOC. C. E. (by letter).—The title of this paper seems to the writer to be a misnomer, for the paper deals exclusively with a very special type of slab, both as regards its support and reinforcement. As an exposition of the peculiarities of a widely exploited—although by no means universally approved—commercial system of construction, this paper is apparently quite complete, yet it seems to contribute little to the solution of the general problem of the flat slab. Mr. Binckley.

Mr. Binckley.

The writer is not a great authority on reinforced concrete, so, even avoiding—as he will—the pitfalls of mathematical controversy, it is with a proper sense of his own temerity that he ventures into this discussion at all. The crushing weight of practical experience and empirical data under which C. A. P. Turner,* M. Am. Soc. C. E., flattened out Mr. Nichols' purely theoretical paper, tends to induce caution in others. Yet the writer has had certain limited opportunities for observation, and has even gone so far as to use reinforced concrete on occasions; hence he feels a modest interest in the subject.

Mr. Godfrey, Mr. Thompson, and Mr. Eckles have all dealt so ably and in detail with certain phases of this paper that the writer will attempt to avoid going over any of the ground already covered, yet he cannot refrain from expressing surprise that the author has accepted and applied the principle of Poisson's ratio to a flat slab of reinforced concrete. The use of this principle in connection with the limited tensile strength of the concrete itself might possibly apply, but it certainly cannot be properly used with the small reinforcing rods.

As regards the effect of compressive stress in two directions, however, the writer believes there can be no question, and in the present case, this may be a factor of real importance. This is especially true of a case where a single bay of a floor system is under test, for the "dishing" tendency of such a bay under load will plainly produce a convergent compressive stress in the upper part of the concrete slab inside the neutral zone surrounding each column head. This compression stress is referred to as "convergent", as it will obviously be greatest at the center of the slab, and its lines will be approximately radial or convergent on this point.

It is not open to question that a homogeneous material, under compression in two intersecting planes at right angles to one another, will show a much higher resistance per unit of area than will be the case if the stress is applied only to two opposite faces. Carrying this one step farther, it must be admitted that a homogeneous body, under compression on all sides equally, cannot yield except to the extent of its own elasticity.

If due weight is given to the greater resistance of the concrete under convergent compressive stress, we have in the present case a reversed "Poisson's ratio." This, though probably without influence on the stress in the steel, would seem capable of producing a notable effect on the observed deflection under load, for the modulus of elasticity of the concrete under conditions of convergent stress would probably be much higher than when the stress is not convergent.

In connection with the discussion of the problem of flat slab design,

* *Proceedings*, Am. Soc. C. E., for August, 1913.

Mr. Binckley.

it would seem pertinent to comment on the all but universal practice of equal spacing of the reinforcing steel in the slabs. It seems to the writer very strange that steel should be placed close to and parallel with a beam supporting one side of a slab, or in fact that equal spacing of steel should be used in any part of the slab. If the steel is parallel and close to a beam, it is obvious that it can be subjected to only a very light stress in tension, and, if above the neutral axis of the beam, may be actually placed in compression. In such a position it can only serve the purpose of supporting shear at its ends, and, for the greater part of its length, is useless.

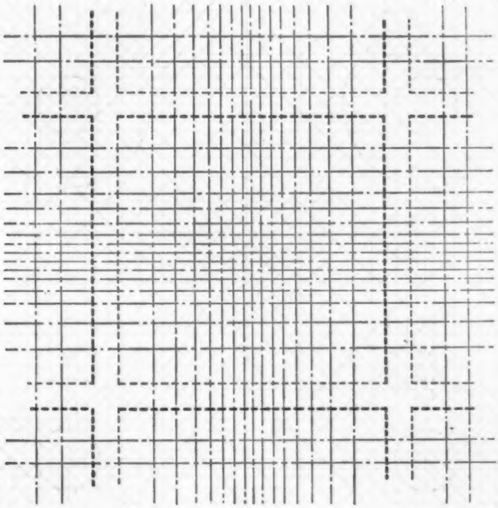

FIG. 21.

The equal spacing of reinforcing steel has always seemed irrational to the writer. A good many years ago he suggested the systems of spacing shown diagrammatically in Figs. 21 and 23. In this system the spacing of the steel is on a logarithmic scale in both directions from the center of the band. In Fig. 21 the two systems of rods are parallel to the beams, which are assumed to be supported by columns at each corner of the bay considered.

It is apparent that, with the arrangement shown, the maximum quantity of steel is at the center of the slab, where the moment is greatest, and decreases toward the sides, so that near the corners of the slab little steel is in evidence. In Fig. 21 the loading of the beams will apparently be severe, and the transmission of the load to the col-

Mr. Binckley.

umns less direct than would be the case if the conventional equal spacing of the steel is followed, so that the arrangement shown in this figure is only suitable for relatively small bays, supported by deep beams. The writer believes, however, that by this system the efficiency of the steel may be nearly doubled as compared with the conventional equal spacing.

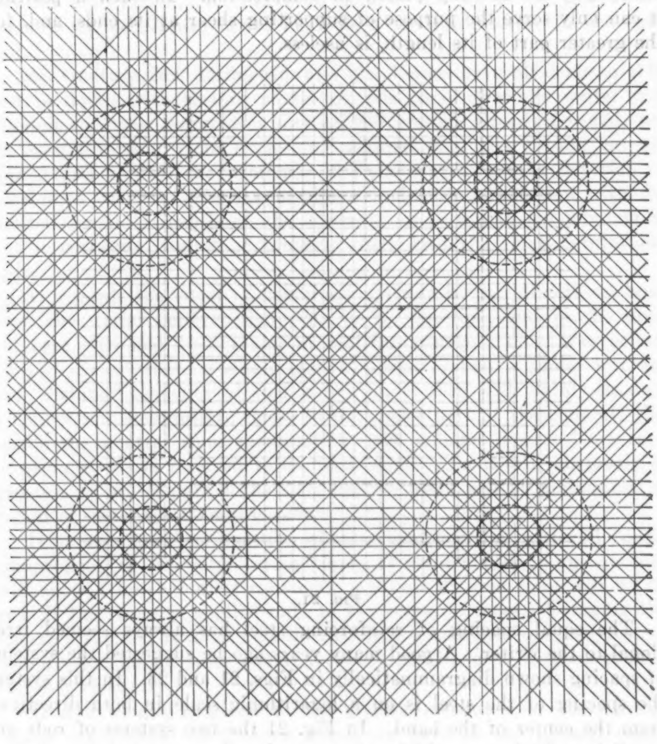

FIG. 22.

In the system suggested in Fig. 23 a notable economy of concrete in the beams will be attained. It will be seen here that, though the same principle of steel distribution—logarithmic spacing—has been used, the loading of the beams is greatly reduced toward the center. The crossed beams at the column heads serve to transmit the load,

already well concentrated, very directly to the columns, while the system of spacing could hardly fail to produce a condition of very much more uniform stress in the steel than is possible with equal spacing. Mr. Binckley.

A certain imperfect perception of this principle seems to have formed the basis of Mr. Turner's mushroom floor design. He provides additional steel in the areas of maximum bending moment—at the

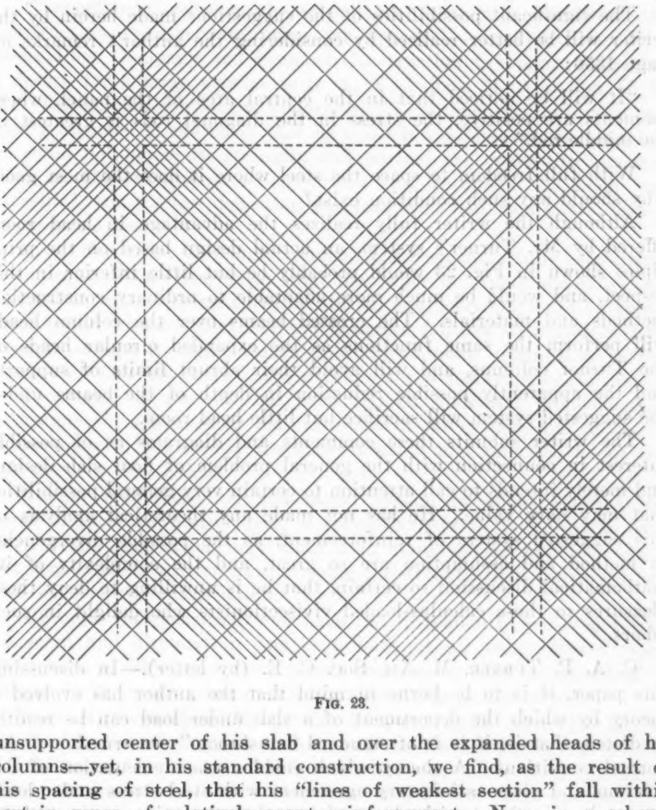

FIG. 23.

unsupported center of his slab and over the expanded heads of his columns—yet, in his standard construction, we find, as the result of his spacing of steel, that his "lines of weakest section" fall within certain zones of relatively scant reinforcement. Now, in a scheme of reinforcement in which such latitude exists for the distribution of steel, it would seem that it should be possible to devise a system under which, theoretically, there would be no "lines of weakest sec-

Mr. Binckley. tion", and the writer ventures to believe that if the principle of steel distribution shown diagrammatically in Fig. 22 were applied to Mr. Turner's floor, such "lines of weakest section" would be relatively hard to find—ignoring in this, however, all questions of shear around the tops of the columns. This Fig. 22 is obviously Mr. Turner's general scheme, except for the change in system of spacing of steel, and the consequent avoidance of the irrational equal spacing and abrupt changes in the quantity of steel shown in his standard construction.*

The significant possibilities of the suggestions made herein by the writer will be better realized by considering the author's remarks on page 1356:

"It will be noticed that in the central area of the panel, where moments are positive, the stress in the diagonal rods is greatest in the middle rod."

With full freedom to space the steel where it does the most good, why should any such condition exist?

Although the writer fully realizes the advantage in head room offered by Mr. Turner's system, an actual design based on the principles shown in Fig. 23 would probably be but little inferior in this respect, and would be much more adaptable to ordinary construction methods and materials. The crossed beams over the column heads will perform the same functions as the expanded circular heads of the Turner columns, and will avoid their abrupt limits of support; and the apparently possible reduction in depth of the beams under the suggested system will sacrifice but little head room.

The writer submits these comments and diagrams as of possible interest in connection with the general problem of floor slab design, and merely intends to call attention to certain very general possibilities that may have value. He has not made any theoretical analysis of this suggested system of reinforcement, as the possible divergencies in method and assumption are so great, and the complexity of its mathematical discussion so certain, that he is unwilling to deny these pleasures to those calculus-loaded vivisectionists who delight in such labors.

Mr. Turner. C. A. P. Turner, M. Am. Soc. C. E. (by letter).—In discussing this paper, it is to be borne in mind that the author has evolved a theory by which the deportment of a slab under load can be readily predicted, that is, if it is of standard "mushroom" construction under normal conditions. A theory which enables exact calculation of deflections and gives satisfactory agreement with steel stress under load, cannot be dismissed with the criticism that there is an error of 30 or 40% in the determination of the stress in a sporadic case of measurement on one side of a bar which was quite likely kinked in

* See page 1353, *Proceedings*, Am. Soc. C. E., for August, 1913.

placing, and the fiber stress—not the average stress, determined by this excellent, but too frequently misleading, method of measurement—which cannot be considered average stress except when proved by its accord with the deflections. Mr. Turner.

As sponsor for the original successful flat slab construction, the writer may say, in answer to Mr. Godfrey, that he has never knowingly submitted it for the adoption of or appropriation by the Engineering Profession at large.

Prospective purchasers, however, have frequently submitted the construction to other engineers for approval. In 1908, the John Deere Plow Company and Leonard Construction Company, submitted a design for a large warehouse to A. N. Talbot, M. Am. Soc. C. E. Mr. Talbot conscientiously investigated a number of buildings and finally reported to the effect that he knew it would carry the load, but he could not figure its strength, and advised the owners to secure a bond of \$50 000 guaranteeing certain test results. The bond was furnished, and the test deflection was just what had been guaranteed—no more, no less.

Since that time, the bond argument has been more fully established as the convincing criterion of what the owner is to get; and if owners insisted on a bond being furnished by all engineers employed by them, there would undoubtedly be fewer concrete failures with loss of life, though perhaps it might result in a considerable diminution in the number of concrete experts and perhaps a few bankrupt engineers.

The term "mushroom" has been applied somewhat promiscuously to designs not approved by the originator of the system, and not built in accord with his standardized specification and practice, as in the case cited by Mr. Eckles.

Having introduced the mushroom system in about \$200 000 000 worth of buildings and bridges, of spans from 12 to 50 ft., in the past seven years, the writer may state that more testing has been done and larger bonds written guaranteeing its strength than for any other kind of concrete construction.

Although inexperienced men have not infrequently removed forms too soon, and although it has been built at all temperatures from 100° above to 24° below zero, Fahr., there is no case on record of a collapse in its construction or a mishap resulting in loss of life.

When it is considered that it has been put up in Australia, India, the West Indies, throughout Canada and the United States, the record of achievement must have behind it something more than mistaken ideas, particularly where deflections under test are successfully guaranteed.

Mr. Thompson objects to Mr. Eddy's treatment in the use of a large Poisson ratio. There is, indeed, some ground for this objection, in that concrete and metal form a heterogeneous combination and do

Mr. Turner. not have properties identical with a homogeneous material. The writer cannot concur in Mr. Godfrey's view, in which he attributes to the direct tensile strength of concrete such wonderful values as to account for test results. Evidently, Mr. Godfrey has never conducted a time or endurance test to determine the basis for his assertion.

The views of both Messrs. Godfrey and Thompson remind the writer of the recent opinion of the Court of Appeals of the Eighth Circuit, in which the conclusion was reached that a reinforced concrete slab was an aggregation, that is, the sum of the functions of the independent or individual parts. Thus, on this basis, its strength should be figured as the sum of the flexural resistance of the rods and that of the concrete as separate respective units. Bearing in mind that the rod reinforcement has hardly bending resistance sufficient to carry its own weight, without deflection beyond a respectable limit, on this theory it could be of no practical assistance to the concrete.

The Court, basing its opinion on the fundamental error involved in the position of Messrs. Thompson and Godfrey, presents a logical *reductio ad absurdum* worthy of serious consideration by the concrete theorist.

The missing link in the chain of logic rendering the opinion erroneous is the disregard of the property added to concrete by the introduction of steel, and this property has never received satisfactory consideration by any writer on the subject; the writer refers to bond shear. It is a neglect of this same property which leads Messrs. Godfrey and Thompson into logical absurdities.

Bond shear is variously defined as adhesion, shrinkage grip of the concrete on the steel, or bond causing the two to act in unison. A shear along the rod, however, must result in the development of indirect stress, which plays an important part in the mechanics of a beam or slab of reinforced concrete.

This shear is equivalent to tension and compression at 45° to the surface of the bar. It depends for magnitude on the increment of stress in the rod, caused by change in value of the true moment.

In a reinforced concrete beam the value of the bond shear passes through zero at the center of the beam, and is a maximum toward the end. The result of this relation is that, in a beam with a small percentage of metal, failure occurs at the center, and in one with a large percentage of metal and having the same form, failure occurs by diagonal tension at a point between the center and the end, induced by these indirect stresses. These bond stresses may be looked on as lines of force emanating from the surface of the bar and following the law of distribution of force through mass, varying in intensity inversely as the square of the distance from the point of origin.

In any structure in equilibrium the stresses are disposed so that action and reaction are equal. The indirect tensions set up by bond

shear, in the case of a beam, are held in equilibrium only by the direct tensile strength of the concrete itself, and hence, while this is intact, as in a newly cast beam for initial small load, a large part of the potential energy of internal resistance is stored up by these stresses, and the steel stress is correspondingly low. As the load is increased, however, the concrete becomes overstrained by the indirect tension of the bond shear, and yields or cracks. The potential energy stored by virtue of the tensions generated by the bond shear is then dissipated, equilibrium is destroyed, and motion results in the form of increased deflection. Increased deflection means generation of new active energy by the downward motion of the load, to be stored up in turn as potential energy in the structure, and the steel element, which has not been overtaxed, stores up this new energy by increased stress, and it is incorrectly stated by the average engineer that the concrete has thrown its burden on the steel. Mr. Turner.

In the flat slab, however, those indirect stresses react on, and are held in equilibrium by, each other, without calling into action in a cumulative manner the direct tensile resistance of the concrete.

Whether we are dealing with a beam or a slab, we must depend absolutely on the bond shear. The beam is accordingly unscientific, because it does not oppose molecular deformation by supplying resistance of the reverse kind to stresses on small particles, as is done in the flat slab of the mushroom system.

Applying the simple fundamental laws of physics to these lines of force, we may work out fully the mechanics of the beam and the flat slab. The correlation of hooping and vertical steel in a column is also covered and explained by these same fundamental laws.

Mr. Godfrey entertains correct ideas about columns, but it would be impossible for him to explain the basis of his good theory of columns without understanding and admitting that his notions about flat slabs are very erroneous.

Verily, as Marsh says, when properly combined with metal, concrete appears to gain properties which do not exist in the material when used by itself, and even the most mystified individual on the subject of the value of the Poisson ratio should be able to understand such simple relations as may be readily developed by a thorough discussion of bond shear.

The laws of action of indirect stress should be noted:

1. Indirect stresses may react through the concrete as a conductor from bar to bar, provided both bars are generating indirect stresses in reverse directions.

2. These stresses cannot react from one bar to another when one bar is under strain and the other is not, because of the law of physics governing the distribution of stress through mass; the stresses, as lines of force, diverge and cannot converge, hence they can only react on

Mr. Turner. each other through the concrete as a conductor enabling divergent lines of force to meet and co-act.

3. The same law limits the efficiency of the action of these stresses through short distances, demanding close spacing and relatively small sections.

4. It follows, from the preceding consideration, that the flat slab differs from the beam in that indirect stresses may react through the concrete from rod to rod without bringing about such accumulative stress on the concrete as in beam design. This element of resistance, therefore, is dependable in the mushroom slab, though it is not in a beam. In the latter a crack in the concrete cuts off all longitudinal or moment-resisting forces in the concrete across the cracked area, throwing the entire burden on the steel, hence, in the beam, we can depend on the steel only as resisting moment. In the mushroom slab, however, a crack merely cuts off a few lines of indirect stress, reacting crosswise from bar to bar, and is relatively insignificant in its effect on the resistance of the mass, as each segment or little area is almost independent of others in the operation of the bond shear stresses reacting from rod to rod across short distances.

5. It follows, from the foregoing discussion, that the Engineering Profession generally is laboring in the dark in concrete design, because it illogically disregards the connecting link between the concrete and metal, thus rating inferior and inherently dangerous construction on a more favorable basis than scientifically designed work. Then, when a failure occurs, some innocuous sawdust or minor defect is blamed for lack of the theoretical and practical knowledge, which should be, but frequently is not, possessed by the self-sufficient engineer.

Suppose, now, we apply the fundamental laws previously outlined to the flat slab of the mushroom system under uniform load, and compare it with a linear reinforcement for the cantilever part.

By Clayperon's theorem, the internal work of deformation equals the product of the mean applied force multiplied by the movement of its point of application.

Let Q equal the quantity of energy stored in the linear cantilever and in the circumferential cantilever with which we propose to compare it.

In the linear cantilever, the potential energy stored in the steel is stored in only one direction, that of the length of the beam. In the circumferential cantilever, however, it is stored both in radial and circumferential directions, and the manner of storage must be investigated before comparison can be made.

Taking a circular section about the columns under uniform load and with unit spacing of bars, if we have a radial deformation, ΔR, we have a circumferential deformation, $2\pi \Delta R$. The radial and circumferential deformations, however, are the same per unit, and there

are 2π units radially in the circumference to one circumferentially, hence the energy stored circumferentially and radially is substantially the same. Mr. Turner.

Although the circular deformations are coincident with and determined by the radial deformations, it is evident that the radial deformations alone determine the vertical geometry of the slab; hence, in the circumferential cantilever, we have half the potential energy stored in such a manner that it produces no vertical deflection, which has a most important bearing in presenting a logical comparison.

It is a well-recognized fact that a board is stiffer and stronger edgewise than flatwise. The beam is an edgewise construction, as compared with the mushroom slab construction, yet the slab construction competes with the beam by a new or different mode of operation, such that a slab having the circumferential reinforcement of the mushroom type is as stiff as a continuous beam of double the depth and the same cross-section of metal.

Let W_1 = the mean load and H_1 the mean deflection of the linear cantilever. Let W_2 = the mean load and H_2 the mean deflection of the circumferential cantilever. Let Q = the potential energy stored up under these loads in the two respective structures, the loads and deflections being such that Q is the same for each, then,

$$Q = \tfrac{1}{2} W_1 H_1 = \tfrac{1}{2} W_2 H_2.$$

Now, as half the energy is stored in the circumferential cantilever in a manner to produce no deflection, we have the relation that $H_1 = 2 H_2$, but as Q is the same for each structure, W_2 must equal $2 W_1$; or, if the deflection were to be the same, then $W_2 = 4 W_1$, or the circumferential cantilever is four times as stiff as the linear cantilever.

This simple mathematical proof demonstrates the absurdity of the beam strip theory presented by the Joint Committee in its report and by Mr. Godfrey in his discussion, compared with true slab theory.

The theory of work may be readily extended to verify Mr. Eddy's scholarly analysis both of deflection and stress, but the writer has not the leisure to present it in this present contribution.

From the preceding discussion, it is apparent that the coefficient used by Mr. Eddy is not a Poisson ratio in the sense that it is a characteristic of an elastic homogeneous material, but it is a coefficient representing* the lateral efficiency of the new property added to the concrete slab by the bond shear. Failure to realize this fact seems to be the root of the difficulty met by some in harmonizing their conception of the laws of statics with plate action.

The plate of reinforced concrete is only an ingenious imitation of the homogeneous article, and its coefficients of necessity are de-

* There could be evidently no lateral efficiency of this property in a beam, because the steel extends only in beam lines.

Mr. Turner. pendent on the connecting link bonding the heterogeneous materials together, producing a true combination and joint action.

It is a perfect or imperfect imitation to the extent that equality of strength throughout the respective tension zones is secured.

The construction shown and recommended in the work of Messrs. Taylor and Thompson is exceedingly deficient in this respect, because the circumferential cantilever of crossed rods does not extend to and beyond the line of contraflexure; hence the circular resistances are inefficient in properly reducing the radial stresses at and about the support, necessitating a large increase in steel at mid-span and developing weakness at the support, requiring for strength a large and ungainly column head.

Mr. Thompson's conclusion, based on experiments on imperfect designs, accordingly, are not applicable to standard construction. He places too much dependence on the absolute reliability of one or two measurements at the cap and just inside the cap. That this is so may be seen from measurements made by no less able an investigator than Professor W. H. Kavanaugh in a carefully conducted test to destruction of a mushroom slab.* In this test two rods in the top of the slab at the edge of the cap showed slight compressions, though all the spokes would indicate that they should have shown tension.

Now, a little kink in the bar, made in handling or under construction, may readily cause just such contradictory results, and sound judgment, accordingly, is a necessity, if results are to be rightly interpreted.

Mr. Thompson's idea that the maximum tension to be borne is greater over the support could be questioned by the writer on the basis of these observations did he place a like dependence on a single abnormal measurement. He prefers, however, not to accept such measurements as scientific and accurate determinations of average stress across the sectional area, there being no check on the opposite side of the bar.

He prefers to trust the practical results he has observed in several thousand acres of this floor in every-day use for years, as affording a criterion of its strength and safety, rather than to depend entirely on extensometer measurements of fiber stresses which occasionally show compressions over columns where the theory of gravitation and the laws of mechanics show that tensions must necessarily occur.

He prefers also to place his confidence in the principle of rigidities, and conservation of energy, as a basis for the correct analysis of the mechanical action of flat slabs, rather than in the opinions of any engineer or any committee who have evidently reached theoretical views not in harmony with these principles.

* Described in *Engineering News* some time ago.

H. T. Eddy, Esq. (by letter).—Mr. Godfrey has taken the opportunity to repeat his objections and warnings against the use of flat slabs, and especially to insist that, whatever may be the true theory of an interior panel of a flat slab of indefinite extent and uniformly loaded, that theory cannot apply either to a single loaded panel or to an exterior panel, and, further, that the theory under discussion involves reliance on direct tension stresses in the concrete. Mr. Eddy.

The full treatment of these questions involves a multitude of details, including bending moments in columns, cantilever action, direct and indirect stresses in concrete, bond shear, shock due to moving loads, the conditions under which beam action prevails over slab action, etc., etc. These questions and criticisms have been raised before by Mr. Godfrey, in connection with the discussion of Mr. Nichols' paper,* and have been answered in detail. Ultimately, dependence must be placed on the facts as they appear in test and practice, rather than on *ex cathedra* statements of what must occur as a consequence of views held on theoretical questions.

The crux of the entire matter seems to lie in the question of direct and indirect stresses in concrete, the latter being those called into play by bond shear and the former by bending.

The indirect tensile and compressive stresses in the concrete due to bond shear enable the rods of the belts which cross each other to co-act with each other under tension and cause the slab to exhibit effects similar to those measured in flat plates by Poisson's ratio. It is evident that as long as the bond is intact at the surfaces of the rods, the indirect stresses—which necessarily have the same numerical intensity as the bond shear—do not exceed the strength of the concrete, and the mechanism for the same kind of interaction exists in the slab as in the plate. This phenomenon, therefore, must be taken account of in one case as much as in the other. Consequently, the value of K for slabs is inextricably bound up with the question of bond shear and indirect stresses in the concrete in the tensile zone. No general discussion of the numerical value of K will be taken up at this time, other than to remark in passing that, in order that a homogeneous solid may remain of constant volume, it is necessary from geometrical considerations to have $K = 0.5$, a value which undergoes greater or less diminution in case of various actual solids, according to their volumetric elasticity.

Likewise, in order that a material sheet or surface may remain of constant area, it is geometrically necessary that $K = 1$, where K refers to lateral distortion in the plane of the sheet or surface, as it does in a slab, and has nothing to do with changes of thickness of the

* "Statical Limitations Upon the Steel Requirement in Reinforced Concrete Flat Slab Floors," *Proceedings*, Am. Soc. C. E., for April, 1918.

Mr. Eddy. sheet. For sheets of various materials, this value of K will be diminished according to its areal elasticity in resisting forces tending to stretch it in its plane.

All reinforced concrete construction is based on the effectiveness and reliability of bond shear. Without its action, no beam or slab could endure, but its action in a slab, not only effects all the results it produces in a beam, but must necessarily cause the slab to exhibit additional properties due to the interaction of multiple-way reinforcement consisting of wide belts of rods in contact with each other. In case the belts are made of numerous parallel rods with a spacing comparable to the thickness of the slab, the slab is sufficiently fine-grained to act very much as if it were of uniform texture but formed of some material not the same as either of its constituents.

It is entirely beside the mark for Mr. Godfrey to argue that, because this composite material has in it reinforcing rods, and Poisson's ratio does not apply to rods considered separately, that this ratio, therefore, has no application to a material like a slab of which rods form an integral part by their intimate union with the concrete, just as truly as if the reinforcement were made into a single sheet and combined with the concrete, although, of course, the value of K would not be identical in the two cases.

What the physical and mechanical properties of such a slab may be cannot well be predicted with certainty, although it may be possible to give a rational explanation of what experiment may show its new and unforeseen properties to be. Whenever such a slab undergoes flexure which causes it to assume a dish-shaped deformation, the peculiar and unexpected properties appear; but it is just on this question of dish-shaped deformation in a slab that Mr. Godfrey has expressed views least in accordance with the facts.

Suppose, for example, a long building with two rows of columns parallel to the long sides, and panels nearly square, so that there are three tiers of panels lengthwise of the building, one long interior tier being between the two long wall tiers. Now, suppose heavy loads to be placed on three panels of a tier or bay extending across from wall to wall. If the writer understands Mr. Godfrey, he thinks that such an arrangement of loading would produce a critical stress in a mushroom slab much greater than would occur if the loading were removed from either two of the three panels. It might do this were it not for the wall supports and the stiff heads; but in fact the reverse is true. The walls entirely prevent the formation of the ends of any such cylindrical trough across the slab, and the stiff heads and walls will tend to accentuate the hollows parallel to the long walls, rather than those perpendicular to them.

The experimental test on which Mr. Godfrey relies to prove his con-

tention has little or no bearing on the slab question, for similar reasons. Mr. Eddy.

The panes of glass on which he experimented* had no continuous support at the edges, such as a slab has, to say nothing of additional local stiffness at the points of support furnishing at these points moments of resistance several times as great as those found elsewhere. The plates were not clamped to practically immovable supports so as to make them act in a manner at all similar to a floor slab such as that under discussion. Consequently, these plates would not act like a floor slab at all, and the assumption that they would is entirely misleading. As just stated, the behavior of glass plates supported in this manner would be the reverse of the slab, in that the troughs in the one would be accentuated in a direction different from those in the other.

This and other matters in the discussions connected with Mr. Nichols' paper directly controvert the statements of Mr. Godfrey, which he evidently regards as incontrovertible, as he has made these statements a number of times and they have never been controverted, as he says; and all this will occur without contravening any law of Nature.

All engaged in concrete design and construction should be equally desirous, with Mr. Godfrey, that it should be conservative and safe, and also that it be adopted where it can safely be done at a cost so moderate as to permit fire-proof in place of combustible construction, as it cannot be if there are unreasonable requirements.

It is not lightly to be assumed that any concern which has been responsible for the design of about two thousand structures of this character without failure, and with inside opportunities for knowing with certainty what can be depended on, and having also enormous financial interests at stake, may not be a good judge of what can and ought to be done in this field.

Mr. Godfrey refers to a paper by the writer entitled "A Comparative Test of Two Full-sized Reinforced-Concrete Flat Slab Panels,"† and tries to convey the impression that the manner of loading and the design of the columns were responsible for the good showing made by the mushroom slab. He states that the test slab was loaded with balancing loads, meaning by that that the load on the over-hang beyond column centers reduced the stress in the central panels so greatly as to make the stresses in the steel small and misleading.

It will be noticed that Mr. Eckles asserts that his experiments showed greatly increased central deflections due to extending the loading into the panels at the sides of the panel tested. These contradictory views cannot both be true.

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* *Proceedings*, Am. Soc. C. E., for August, 1913, p. 1361.

† *Engineering News*, March 27th, 1913.

Mr. Eddy. The truth is that neither of them represents the facts. If there is anything that has been established by numberless load tests of mushroom slabs, it is that in the standard mushroom slab the central deflection and stresses of an interior panel are not affected to any appreciable extent by the loads on surrounding panels. A very small effect has been thought to be discoverable at the centers of panels situated diagonally, but no mutual effect between panels beside each other.

This fact, which would at first sight seem inexplicable, is apparently the result of a close balancing of two opposing effects, *viz.*, any load just outside a panel will have a tendency to lift the center of the panel, but, at the same time, it will tend to increase the deflection of the side belt between it and the center of the panel. Any such deflection at the side belt will have a tendency to increase the deflection at the center of the panel. Whether the lifting or the increase of deflection preponderates depends on the relative rigidities of the belts and the slab and column heads, with the result stated above. Not having at disposal sufficient data in other types of construction to state conclusions with certainty, the writer has refrained from doing so.

The panel tested by Mr. Eckles was at a corner of a slab. It may have been a mushroom slab in general design without having any features of the system in this corner panel beyond one mushroom head at one of the four corners. More details are needed in order to show that the system is in any way responsible for the results of Mr. Eckles' test. The writer has not attempted to give an analysis for such a panel. It is customary to design wall panels differently from interior panels, but that is no argument against the approximate correctness of the analysis for interior panels.

To return from this digression to Mr. Godfrey's criticism that the results of the test slab were due to balancing loads on the over-hang: The tables and diagrams of the test show that in Loads 3, 4, 5, and 6—which were the ones particularly relied on for showing the behavior of the slab—the loading resting directly on the panel was more than twice as great as that on the entire surrounding over-hang, and, in the case of all the other loads, that on the panel itself was greatly in excess of that on the over-hang. These, consequently, were not balanced loads, and besides, there was no loading whatever on the areas above the column heads for Loads 1 to 6, inclusive, and no large loads later. The loads were applied over an area in the form of a Greek cross, with the intention that whatever load might be applied would be effective in causing bending moments and be situated so as to be less favorable to the mushroom slab than to its competitor.

The columns were 18 in. square and 5 ft. 6 in. from top of footing to bottom of slab, and not, as Mr. Godfrey asserts, "about half as thick as their height." Their diameters, as well as those of the mushroom heads, were in the usual relation to the span. This structure cor-

responded to a basement floor slab, but, because there were no upper floors, the columns necessarily had very small load stresses per square inch. That, however, did not affect the resistance the columns would afford to unbalanced moments, except to render the columns subject to actual tensile stresses in their reinforcing rods by reason of the small superimposed load. The columns were at a disadvantage for this reason and also because the unbalanced bending moment in the slab was not resisted by the stiffness of columns extending upward as well as downward from the slab, as would occur in a building. That may possibly be regarded as partly offset by the fact that the columns were only 5 ft. 6 in. long, but there was no such divergence between usual conditions and those of this test as Mr. Godfrey asserts. Mr. Eddy.

Nevertheless, this test was not made (as seems to be assumed) to demonstrate the strength of this form of construction, *per se,* but to make a comparative test between it and another form of construction, and to demonstrate that though one operated on the principle of the beam the other did not.

The writer had nothing whatever to do with the design of the slab or the arrangement or carrying out of the test, except to suggest the Greek cross as the ground plan of the loading. The data obtained were submitted to him for discussion. Mr. Godfrey or any one else has the opportunity to discuss them and come to any conclusion the figures will support. Innumerable load tests of standard mushroom slabs have been made. This test was not made to find out anything about the elastic behavior of mushroom slabs, about which there was any real doubt or lack of knowledge by those designing these structures, but to put the facts in evidence, and further to establish their manner of failure when stresses are carried beyond the yield point. This last was a most important piece of evidence which had not heretofore received such experimental certification as to settle it beyond dispute.

It would seem on the face of it as if this test on a full-sized slab, in which all the details are carefully tabulated by reliable observers in such form as to admit of complete comparison with any other tests, might at least afford as good a foundation on which to base conclusions as the experiments made by Mr. Godfrey on a couple of glass plates under conditions not in accordance with those obtaining in floor slabs, and yet Mr. Godfrey intimates that absolute dependence is to be placed on the indications of his glass plates, and that the slab test should be disregarded as untrustworthy.

It may be that Mr. Godfrey believes that this test "has not even a theoretic interest", but he may find it difficult to persuade any to agree with him except those who are so wholly blinded by prejudice or interest as in fact to be unwilling that any reinforced concrete struc-

Mr. Eddy. tures shall be built for use unless designed in such an innocuous manner as to be "out of the running."

Mr. Godfrey has also repeated his unsupported statements as to the unreliability of reinforced concrete construction under rolling or jarring loads, and seems to regard his mere assertion as sufficient to settle the question in this case also. In order to show how groundless is this opinion, which apparently has no other foundation than his own say so, the writer would quote from W. K. Hatt,* M. Am. Soc. C. E., as follows:

"It is well known that concrete, because of its lack of elasticity, absorbs or deadens vibrations, and the sound caused thereby. It is not probable that vibrations reach the steel. The speaker has knowledge of many experimental attempts to loosen the bond by shocks and vibrations. So far smooth bars encased in concrete that have been subjected to shocks and long-continued vibrations seem not to have lost any of their original strength of bond. Likewise the concrete on the compression side of a reinforced concrete beam that has been loaded and released from load some 2 500 times to high working stresses seems not to have been substantially weakened thereby."

For, far as these experiments go, they give no ground whatever for Mr. Godfrey's statement, into which he would seem to have been led by applying a line of reasoning derived from the behavior of all-steel structures which is inapplicable to reinforced concrete. An impact or blow at any point of a steel bridge is propagated longitudinally along elastic members extending linearly from the point, and it goes practically undiminished to the farther ends of those members, where it is subdivided among other members and propagated still farther. An allowance of 80 or 90% is usually added for impact to the static effect of a moving load.

Impact, or the effect of a blow at any point of a reinforced concrete slab, however, is entirely different from this. In the first place, the effect of the blow does not travel in one direction only, but in all directions radially from its point of application, so that in a very thin slab, its effect at any other point would be inversely as the distance, and in a very thick slab inversely as the square of the distance. This would make the allowance for impact in a slab very small.

Secondly, the effect of impact must be inversely proportional to the weight of the body receiving the blow. It may be assumed that steel costs from twenty to thirty times as much per ton as reinforced concrete, and the effect of impact on a steel structure costing the same as a slab would be from twenty to thirty times as much, for that reason.

Thirdly, the continuity and stiffness of the slab greatly reduce its vertical, lateral, and torsional deformations below those of a steel

* *Proceedings*, Nat. Assoc. of Cement Users, Vol. III, 1907, p. 60.

structure. The work done during impact, and its effect, depends on the amplitudes of the deformations. In particular, the horizontal resistance of a slab is many thousand times that of a steel structure. The vibratory energy absorbed by a slab during an impact is consequently small. Mr. Eddy.

Fourthly, the small amount of energy which is absorbed is not transmitted (as it is in a highly elastic and resilient structure) to a considerable distance in the slab, but, owing to the nature of concrete, is dissipated near its source, transformed into heat, and rapidly absorbed.

Fifthly, slabs are tough, and not brittle, like terra cotta, for example, so that, in cases where great weights have fallen on them, little effect has been produced, whereas brittle slabs have been smashed under such circumstances, and have failed.

The concrete of the compression zone is such a shock-absorber as to protect the tension zone from jarring and vibration, both as regards steel in tension, and concrete as well.

For all these reasons the shock which a rolling load imparts to a slab is inconsiderable, and is absorbed and dissipated so readily that it is a negligible factor, instead of being such an important and overshadowing consideration (as Mr. Godfrey says) as to prevent its safe use in railroad viaducts and the like.

In support of these contentions the facts respecting the concrete railroad bridge, Fig. 24, which carries the track of the Soo Line between Chicago and Minneapolis over Wilson Street at Amherst, Wis., are instructive. It is a slab or girder, 18 ft. 6 in. wide and 2 ft. 3 in. thick, with reinforcement in excess of 1%, acting largely as a girder, but with some slab action, having a central span of 25 ft. 9 in. and designed to carry the heaviest locomotives with inappreciable deflections. It would be extremely difficult to cause failure by any loading that could be placed on it. It would require a load of more than 600 tons on this panel to cause a unit stress in the steel of 16 000 lb. This bridge has given such excellent satisfaction for years that the Superintendent of Bridges and Buildings of the Soo Line considers reinforced concrete as the desirable material for such spans.

Particulars respecting other satisfactory slab bridges carrying street-railway traffic are to be found in the writer's book.*

It would seem that Mr. Godfrey's opinions respecting the availability of reinforced concrete for such purposes are not in accord with successful practice, and are not such as should be expressed with vehemence until they are supported by careful experimental tests, rather than by vague references to failures which are used merely to cast discredit on reinforced concrete design in general. There have been lamentable failures of all-steel structures, but that is no reason

* "The Theory of Flat Slabs," pp. 67-69.

Mr. Eddy. for discarding them. It is reason for discriminating analysis and careful design. The same lesson is to be derived from failures of reinforced concrete design, rather than to use them as a pretext for indiscriminate condemnation of reinforced concrete as unsafe for moving loads.

Mr. Eckles must have made some mistake in ascribing to the writer any previous attempts to treat reinforced concrete flat slabs. It is true that a paper on "Thin Homogeneous Circular Flat Plates" was published by the writer some years ago, but it contained no reference to any application of the flat slab theory. In the paucity of theoretical developments which could possibly furnish the basis for slab theory, this paper was used again and again in current engineering literature for that purpose, but the writer was in no way responsible for this, and in fact found so much in these applications with which he was at variance, for one reason or another, that he was induced to investigate the theory of reinforced flat slabs, as he has done in his book. The attempt now under consideration is his first effort in that line, and is entirely independent of his previous paper. When Mr. Eckles mentions discrepancies between the writer's former theories, and conclusions based on facts, it would be well to cite references, as the writer cannot recall any work of his to which these remarks apply. The only development of slab theory in any work of his is in the one on "Flat Floor Slabs," in 1913, and the paper now under discussion is the application of the theory there developed to test results in several slabs. There is certainly no discrepancy here, such as Mr. Eckles would have us believe.

Mr. Thompson apparently does not understand the object the writer had in view in his paper. It was to show that test results as to deflections and stresses, as far as they were available, were in such accord with the analysis which he had proposed in his book that a reasonably close practical agreement existed between them, especially in the case of the mushroom system with which he was in a position to obtain inside information more complete than was the case with other systems. At the same time he utilized for comparison such information respecting other systems as was at his disposal.

Mr. Thompson seems to think that if he could show that the test slab of the Northwestern Glass Company's Building was insufficiently reinforced, or over-strained by the test load, or otherwise incorrectly designed, he would thereby discredit the computations in the paper; but such is not the case.

It is no doubt the truth, as Mr. Thompson alleges, that it is possible to draw further conclusions from the test data given than were drawn by the writer, and it may be surmised that the four conclusions stated by Mr. Thompson at the beginning of his discussion do not by any means exhaust the list. It is assumed that wall panels must necessarily

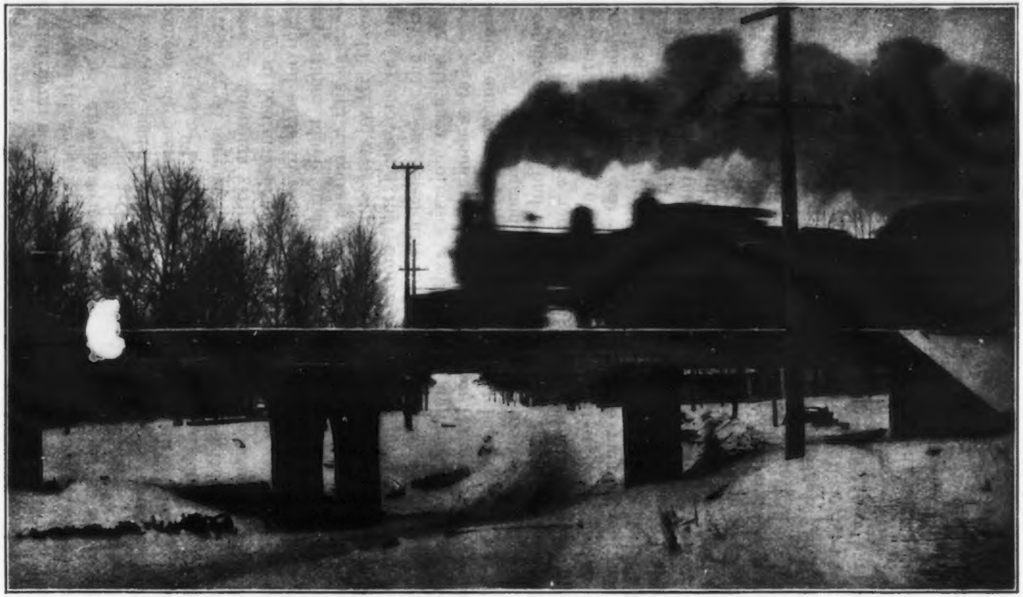

Fig. 24.—Soo Line Railroad Bridge Over Wilson Street, Amherst, Wis.

differ in design from interior panels, and that every one conversant with slab design is perfectly aware of this, and hardly needs to have it called to his attention. It is believed that wall panels invariably differ in this way. The amount of that difference, however, is dependent to a large extent on the character of their connection with the walls and pilasters. Mr. Eddy.

The deflections of Panel A, shown in Fig. 7, when compared with those in Panel D, do not indicate that in this case the moment of resistance of Panel A as actually designed needed to be increased by more than a slight percentage to make its resistance equal to that of Panel D. As this is a question not yet entirely amenable to analysis, it remains to a very considerable extent to be determined in each case by the judgment of the designer. Other types of slabs having more flexible connection with the columns would no doubt require greater additional stiffness in wall panels than was needed in this case.

The principal questions at issue, however, as seen by Mr. Thompson, appear to be based on his two assertions that Equations (*a*) and (*b*) do not in fact give results that accord with tests, and as the principal support of that assertion, the further assertion that the observed stresses at gauge line 111 were not abnormal, but such as to be expected. The writer will proceed to meet these assertions at some length and then treat other matters broached by Mr. Thompson.

It should be stated first that the writer had nothing whatever to do with the design or testing of this slab, nor with that of any of the other slabs treated in this paper, and it is a subject of regret with him that the number of gauge lines over the column heads was so small, and that they were so irregularly distributed. However, he was compelled to make the best use possible of the data at his disposal, and he is still of the opinion that his interpretation of the results is substantially correct, as he will proceed to show by making use of analogous data from an unpublished test which has been put at his disposal since the paper was written.

Additional Data.—This is the case of a slab, 7 in. thick, with panels 18 ft. square, design load 150 lb. per sq. ft., and loaded at once over four panels forming a square, with a load which if uniformly distributed would have an intensity of 151 lb. per sq. ft., but disposed so as to leave several open spaces. It is estimated that the stresses in the steel over the center column were such as would be produced by a uniform load somewhat in excess of the design load. The four loaded panels were two wall panels and two adjacent interior panels, just as in the Glass Company test.

Fig. 25 is a plan of the central column head and the slab rods crossing it, with the outer ring rod 8 ft. in diameter, the inner ring rod 4 ft. 6 in. in diameter, and the cap 3 ft. 4 in. in diameter. The plan also shows the relative positions of the numerous 8-in. gauge

Mr. Eddy.

lines, most of which were in the semicircle most distant from the wall, and the length of the laps drawn to scale. Each belt consisted of thirteen ⅜-in. round rods, except the direct belts which cross the wall panels from pilaster to column, and these had sixteen rods each.

The measured stresses, with the mean embedment of the axis of each rod at the extremities of the gauge lines, are given in Table 16.

Gauge lines 14 and 44, were on the top and bottom of the same rod, as were 15 and 42 also, from which it is evident that the other observed stresses on the tops of rods are most certainly in excess of the mean stresses to which the rods were subjected. Especially must this be true at the edge of the cap, where there is a sudden change in the flexibility of the slab. For that reason it is quite uncertain what actual mean stresses are indicated by the stresses which were observed on the upper surface of the rods crossing the edge of the cap radially; but we may be sure that they are much smaller than those corresponding to observed elongations.

It thus appears that some part at least of the excessive stresses apparently found here are due to placing the gauge lines on the upper surface of the rods, and the explanation which has been offered for these excessive stresses as being due to the thick boss formed by the head is needed as a reason to account for only a fraction of the observed results.

Equations (a) and (b), proposed for computing the maximum stresses in the slab rods of side and diagonal belts, respectively, in the head, give results which do not differ greatly from each other.

Let $W = 50\,000$ lb., $L = 216$ in.

$A = 13 \times 0.11 + = 1.4365$ sq. in.

$d_s = 5.5625$ in.

Then Equation (a) becomes

$$f_s = \frac{50\,000 \times 2 \times 216}{800 \times 5.56 \times 1.43} \left[\frac{3\,(216)^2}{(216 - 40)^2} - 1 \right] = 12\,000 \text{ lb. per sq. in.;}$$

but $L'^2 = 2\,(216)^2 - (96)^2 = 102\,528$, and $B' = 265.4$, and Equation (b) becomes

$$f_s = \frac{50\,000 \times 216 \times 3.37}{400 \times 5.56 \times 1.43} = 11\,500 \text{ lb. per. sq. in.}$$

The reason that these computed values are in excess of those observed, lies in the quantity of steel to be found in the laps, as shown in the plan, which quantity is evidently largely in excess of that for which the formula was computed, as will be seen on referring to page 32 of "The Theory of Flat Slabs," where it appears that the mean cross-section of the slab steel was assumed to be 4.25 times the area of a single belt. The case in hand certainly exceeds this, as all the

belts are lapped as far as beyond the inner ring rod, with a consequent reduction of the stress at the edge of the cap of perhaps 25% below the value computed by Equations (*a*) or (*b*). Mr. Eddy.

It should be noted that the observed results show some diminution of stress with the embedment, but, in the slab rods themselves, this is by no means proportionate at the edge of the cap to their approach to the neutral axis, although it is more closely proportionate elsewhere.

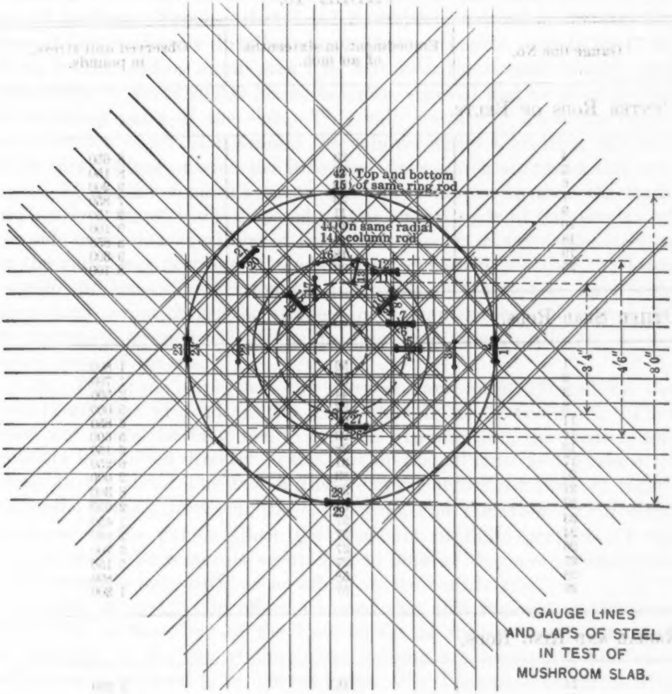

FIG. 25.

All these gauge lines were located around the central column corresponding to Column 45 of the Northwestern Glass Company Building. Besides these gauge lines, seventeen more were observed at three other column heads corresponding to Columns 36, 37, and 44 of Panel D, on the edge of the loaded area. The gauge lines, with one or two exceptions, were located (as were those around the central column) so that the edge of the cap nearly bisected each of them. The largest pair of stresses observed showed 7 400 and 7 700 lb. per sq. in., respectively,

Mr. Eddy.

being on parallel adjacent diagonal laps. The next smaller observed stresses were 6 450 and 6 250 lb. per sq. in., respectively, with no other values as large as 6 000. In general, the values of the maximum stress in these gauge lines varied from 70 to 80% of those over the head of the central column. The two first mentioned appear to be abnormally large, for some unknown reason, as others similarly situated were much smaller.

TABLE 16.

Gauge line No.	Embedment, in sixteenths of an inch.	Observed unit stress, in pounds.
Center Rods of Belts.		
4	23	8 650
5	23	8 150
6	23	9 200
7	23	7 850
9	11	9 150
10	11	9 100
18	18	8 850
19	18	9 300
25	30	8 100
Other Slab Rods.		
1	56	1 800
2	56	1 700
3	44	3 500
8	27	5 000
11	22	6 850
12	23	5 600
13	28	6 150
17	31	6 250
20	33	2 000
21	33	3 300
22	57	2 650
23	57	1 450
24	57	1 000
26	27	5 700
27	27	5 150
28	58	800
29	57	1 300
Radial and Ring Rods.		
14	48	3 350
44	......	2 300
15	57	1 850
42	......	550
16	33	3 750

Again, in Mr. Lord's report of the test of the Deere and Webber Building,* Fig. 14 shows cracks on the upper surface of the slab within an inch or so of the edge of the caps in the loaded area; but these cracks cannot be said to lie more outside than inside of the edges of

* *Proceedings*, Nat. Assoc. of Cement Users, Vol. VII, 1911.

the caps. It is clear, therefore, that those radial gauge lines that cross the edge of the cap in Fig. 25 will unquestionably show maximum steel stresses. Gauge lines 13, 14, and 44 give no indications that maximum stresses are to be found outside the cap. These additional data are thus seen to be confirmatory of the substantial accuracy and applicability of Equations (*a*) and (*b*). Mr. Eddy.

Recalculation of Equations (a) and (b), in the Northwestern Glass Company Building.—Consider now the effect of the distribution of the actual loading. Suppose that Load 3, which consisted of 380 000 lb., were at first uniformly distributed over the four panels, each 16 by 17 ft. Next, let the loading on an area 7 by 8 ft. over the central column, and amounting to about 20 000 lb., be entirely removed and be piled on the surrounding parts of the slab. This will practically leave the cantilever portion of the slab around the column without loading, and will place this loading outside the inflection lines which surround this column. If similar removals occur at other columns, so that the loads are nearly symmetrically disposed on the central parts of the panels and the side belts, then it will be of little consequence, as far as the stresses in the cantilever portions of the slab are concerned, whether the loads outside the lines of inflection are uniformly distributed or have wide open ways across them; the stresses in the cantilevers will necessarily be increased by removal of the loading from the cantilevers to the other parts of the slab.

Mr. Thompson says that the stresses "are more affected by the [total] amount of load in the panel than by their position". That certainly is not the fact, for, to take an extreme case, the loads might possibly be applied directly to the tops of the columns and produce no stress in the slab. The fact is that removal across the lines of inflection affects the stresses in the cantilevers, and removals within other portions of the panels affect the moments in those portions of the panels where the removals occur, but so long as they are symmetrical, such removals have little or no effect on the cantilevers.

In the paper it is in effect assumed that this removal added more than 10% to the effect of the load, so far as the stresses in the steel at the edge of the cap of the center column are concerned, and such allowance is believed to be not too great. Mr. Thompson disagrees with this on general reasoning, based apparently on principles applying to beams simply resting on supports, instead of firmly fixed to stiff supports, in which the position of the lines of inflection would be affected by the distribution of the load to an extent which is not the case in slabs, where the lines of inflection are practically fixed in position and remain the same for different distributions of load. Mr. Thompson, however, apparently states the truth inadvertently when he asserts that "the same load uniformly distributed would not have caused larger stresses than was caused by the load placed as shown in Fig. 5", for,

Mr. Eddy. as we have just seen, it would certainly have produced smaller stresses.

In case the lines of inflection are practically unaffected by shifting the load from the cantilever, as just described, the stresses at the edge of the cap will be increased, and it will also be justifiable to neglect the effect of any change in distribution of loads due to the center aisles, 18 in. wide, when treating the cantilevers, though perhaps not in treating other portions of the slab.

On repeating the calculation of the maximum stress at the edge of the cap, as given in the paper, the figures were found to be somewhat inaccurate by reason of using an incorrect value for the clear span between the caps. The correct figures are:

Diameter of cap = 50 in.; diameter of head = 87 in.

$L_1 = 204$ in., $L_2 = 192$ in., $B_1 = 204 - 50 = 154$ in.

$L'^2 = (204)^2 + (192)^2 + (50)^2 = 86\,049.$

$L^2 = (204)^2 + (192)^2 = (280)^2.$ $B' = 280 - 50 = 230$ in.

If $W = 106\,250$, $d_3 = 6.5$ in., and $A_1 = 15 \times 11 + = 1.6567$ sq. in.

Then, by Equation (a), $f_s = 20\,700$;

and by Equation (b), $f_s = 19\,500$;

which values may need to be increased somewhat for lack of laps, as previously stated.

Therefore, a mean value of 20 000, say, must replace the erroneous value, 17 000, originally given.

It would be interesting to see the calculation by which Mr. Thompson says a value of $f_s = 15\,400$ is obtained by the application of Equation (b). The only change which he has suggested in the computation consists in scaling down the load of 106 250 lb. per panel, which he says is too large, though that does not appear to be the fact, as has already been shown. However, proceeding on that basis for the moment, let the load be scaled down to 96 000 lb. per panel, then, provided the originally computed value given in the paper ($f_s =$ 17 000 lb.) were correct, we should find $f_s = 15\,400$ lb. Recalculation, however, makes the value of f_s at least 19 500 instead of 17 000 lb., and the scaled value would consequently be 17 600 instead of 15 400, a discrepancy of 2 200 lb. per sq. in., or more than 14 per cent. It must be that Mr. Thompson did not actually compute this by the formula, but accepted without investigation the erroneous figure in the text.

There were, in fact, no laps whatever in the slab steel over Column 45, and the general formula, as discussed on page 32 of "The Theory of Flat Slabs," would need to be modified slightly for that reason, so that, in applying Equations (a) and (b) to this case, the reduction of the cross-section of the steel in the belts below the mean value assumed in establishing the formula would require a corresponding increase of the maximum stress at the utmost of, possibly, 20% above that com-

puted from the tentative coefficients in Equations (*a*) and (*b*). Indeed, 10% in addition to the computed stress is sufficient to cover the abnormal unit stress of 22 000 lb. at gauge line 111. Mr. Eddy.

It may truthfully be said, therefore, that there is good agreement of Equations (*a*) and (*b*) with stresses in this test, even if the actual stresses be assumed to be considerably in excess of any observed.

Abnormal Stresses at Gauge Line 111.—Mr. Thompson has misunderstood the facts stated in the paper respecting the stress at gauge line 111, since he has confused the elbow rods with the slab rods. The eight radial elbow rods, shown in Fig. 15 under the slab rods, are of $1\frac{1}{8}$-in. round steel, and are of great stiffness compared with the slab rods. As stiffness is proportional to the fourth power of the diameter, one of these rods is more than 80 times as stiff as a slab rod. These are the rods that were forcibly pried and held in position in the slab until relieved by the application of the test load, when they acted with that load in inducing stresses in the slab rods which they were unable to produce while acting alone, being held by the shrinkage stresses of the concrete. Thus, after the slab had once suffered flexure by the application of a considerable load, and had had a chance to recover, the radial rods and slab rods would come back to a position of equilibrium and mutual adjustment different from the one they occupied before the application of the load. A renewed application of loading would then show elongations and stresses measured from this new position of equilibrium corresponding to the loading. If concrete was a perfectly elastic material, however, such an action would not occur. If any such steel stresses were around the edge of the cap at Column 45, as Mr. Thompson states, they could not fail to show at such a gauge line as 107, which crosses the edge of the cap. The assertion that the stress at 111 was not abnormal because it increased and decreased with the load, is inconclusive, because that depends on how much decrease occurred on the removal of the load as compared with the previous increase. The fact is that the decrease of stress at 111 on the removal of the load was so small as to make it apparent that some such action was occurring as has been described. That this is the correct explanation of the observed abnormal stress at gauge line 111 seems to be established conclusively by the observed unit stress of 11 600 lb. remaining after the removal from Panel C of all but 7 000 lb. of the load of 47 100 lb., and an observed unit stress of 10 300 lb. still remaining a day later. This abnormal stress after the removal of the load, such as is not shown at any other gauge line, makes it clear that the stress at 111 was abnormal before the load was removed, and nullifies any reasoning that leaves that fact out of account.

Location of Gauge Lines.—A misconception is likely to arise respecting the matter of stresses inside the cap and outside of it, though disregard of the fact that absolutely sudden changes of stress cannot

Mr. Eddy. occur at any point of a rod, because such changes must be more or less gradual from point to point. Owing to this fact, and to the fact that the edge of the cap is comparatively thin, large stresses are to be found for some small distance inside the geometrical outline of the cap, and are not confined to the area strictly outside of it. Indeed, some designers, in order to take account of this, have assumed the effective diameter of the cap as 2 or 3 in. less than its actual diameter. This question has its bearing on the location of the gauge lines with reference to the geometrical edge of the cap.

Again, the single abnormal result cited by Mr. Thompson, and shown in his gauge lines, *A* and *B*, does not accord at all with what is generally known respecting the distribution of stresses at the edge of the cap, and, until more specific information is available, it should be disregarded because a single measurement of this kind cannot be depended on, even were it not known that it is abnormal. It would be quite understandable that whichever of Mr. Thompson's gauge lines, *A* or *B*, lay across a crack would show the greater stress. As seen in the Deere and Webber test, however, that might as probably occur with *A* as with *B*; or it may merely be due to a so-called kink in the rod being straightened out, or merely the extra tension in the upper surface of the rod.

Mr. Thompson's attack on the validity and general accuracy of Equations (*a*) and (*b*) is thus seen to have a very slight foundation in test data, and his estimate that the actual stresses may exceed those computed by these equations by as much as 45% may be disregarded.

Mr. Thompson says, respecting Mr. Turner's test slab, that it was broken by loads placed on the center panel, with comparatively little load on the projections. That is the truth, Mr. Godfrey and his balanced loads, to the contrary, notwithstanding. Mr. Thompson criticizes the test because it was not directed toward elucidating the particular problem in which he is interested. Mr. Turner, however, had in view the solution of a different problem, for which he was willing to expend money. Neither was that problem the one Mr. Godfrey would like to have seen investigated. It will be interesting and valuable for these gentlemen to investigate the problems they have in view, but they can hardly expect others to put aside their own pressing problems because they themselves are not interested in them.

It may be stated again, however, that, so far as can be learned from numerous load tests of mushroom slabs, central deflections of panels are unaffected by loading adjacent panels, which, if true, may permit this test to be used to solve some of the problems proposed by these critics. However, it is perfectly evident, from the report of the test and the photographs of the failure,* that Mr. Thompson is mistaken if he supposes that the steel and concrete just at the edge of the cap

* "The Theory of Flat Slabs," p. 92.

were ultimately more severely stressed than at points somewhat farther from the center of the columns, for the test slab gave way at points at least as far distant from the cap as gauge line 103, instead of at the edge of the cap, and this was a case of belts without laps. Mr. Eddy.

Mr. Thompson entirely misses the point of the argument, respecting the comparison of the stresses at Column 36, and those at Column 46, which are similarly situated with respect to Load 3. At 36 the stress lines given are those largely due to the loads in Panel C, and at 46 to loads in Panel D. Whatever changes occurred at 46 by increasing the load on D might be expected to be paralleled, were the load on C increased. What is there "absolutely misleading" about this? From the behavior at 46 when the load on D was doubled, it might perhaps be surmised what would happen at 36 were the load doubled on C, which it was not. What misleading conclusion has been here advanced?

Certainly there is nothing so misleading in this as in Mr. Thompson's statement that the slab bar on which gauge line 103 was taken "was one of the outside ones of the diagonal band". In fact, however, the bar was the third one from the center bar and the outside ones were the seventh from the center bar, all equally spaced, and so this bar was not half way to the outer bar, where it was asserted to be by Mr. Thompson. That, unquestionably, is "absolutely misleading".

Mr. Thompson criticizes the values obtained for stresses in the steel in the diagonal and rectangular bands because he says they "are calculated from formulas derived without regard to the size of the column head, and, * * * would apply to a construction with a column head equal to a sharp point and one of any size whatever." He says "this assumption, on the face of it, is erroneous". Whether erroneous or not, one thing is certain, *viz.*, the stresses, if correctly computed on the assumption stated, will not become larger on account of making the head of appreciable size. The effect of the head, if anything, will be to decrease the calculated stresses at mid-span, and the error, if such it be, will be on the side of safety, and on that account is unobjectionable, from a practical standpoint, however much it may detract from the accuracy of the formulas. As the criticism is based on what appears "on the face of it", it might be well for Mr. Thompson to make an estimate, if he can do so, as to the amount of the effect produced at mid-span of these belts by ordinary sized heads, when due consideration is given to the fact of the comparative fixity of the lines of inflection, thereby conferring on the central parts of the panels outside the cantilever a semi-independent character, which justifies the assumption as one approximately correct. The final proof of the applicability of the formulas, however, is to be found in their accordance with test results, as has been shown in the discussion of the tests.

Mr. Thompson criticizes Equations (a) and (b) because the observed

Mr. Eddy. stresses at the edge of belts opposite column centers fall short of those computed from these equations. It must be remembered that these equations were derived on the supposition that the steel was as near the top of the slab as it is over the column caps. As it is in fact at a much lower level at the edges of the belts, less stresses will be observed here. This is part of the reason for the occurrence of large stresses at the edge of the cap; but the theoretical stresses thus computed furnish a basis on which to compute maximum stresses in slab steel over the head.

Mr. Thompson objects to the word "theoretical" in this connection. The word correctly designates a result reached from a consideration of the predominating principles involved. It does not mean, when applied to engineering structures, that all factors have been introduced into the premises. It means such simplified assumptions and premises as will take account of the dominating factors and give as a result a reasonable approximation which is accurate enough for practical purposes. As neither Mr. Thompson nor, so far as is known, any one else has proposed an approximate theory in any practical agreement with experiment, it would seem to be the courteous thing not to assail a proposed theory on the basis of assertions which seem not to be well founded.

Mr. Thompson says that a design, such as that of the Northwestern Glass Company Building, does not have a proper factor of safety, the inference being that radical changes of design are required, whereas it appears from the preceding analysis and tests that all that is required to bring down the unit stresses in the steel at the edge of the cap, and reduce them by any desired amount, is merely to place additional short rods across the cap between the slab rods, of a length from three-tenths to four-tenths of the span, and no general alteration of design is required, as is intimated.

Mr. Thompson refers at the same time to stresses in the concrete. It is not the object of the paper to go into that subject further than absolutely necessary, and it will be postponed for future consideration, with the remark in passing that, where an absolutely flat slab is not required, every one is aware that increased thickness of slab in the vicinity of the cap will reduce the concrete stresses.

What the writer is especially concerned about in this discussion is, not the close accuracy of the particular computations of this paper (for they are admittedly based on average data which do not necessarily coincide altogether with the particular examples to which they are applied), but to convince those interested in slab construction of the fact that a method has been found for making a rational analysis of slab design, a method which will ultimately enable the engineer to predict results with somewhat the same certainty as is now attained in ordinary structural work. That may require modifications of constants, etc., in his practical formulas, which apply to the mushroom system.

Such modifications will be no greater than every engineer uses in bridge construction, where a method is used which is not expected to result in the same final formulas for stresses in all types of bridges. Mr. Eddy.

This paper proposed to discuss "Steel Stresses in Flat Slabs," and see how closely they could be computed by the analysis published by the writer in his book. Mr. Mensch regards the paper as taking up the general defence of the mushroom system. That is an unwarranted enlargement of the scope of the paper. He seems to think that if he can make it appear that any of his numerous objections to flat slabs are admissible, he has thereby cast discredit on the paper in some way. Let it then be said, by way of preface, that Mr. Mensch has not called attention to a single discrepancy between the computed and the observed results. The only discrepancies mentioned by him are between results given and confirmed both by analysis and experiment with what he believes they should be.

The real question at issue, however, is this: can the writer actually compute steel stresses in any one type of flat slabs correctly? We are led to believe that Mr. Mensch thinks he cannot do this, but, instead of meeting the main issue directly, he introduces a number of side issues. The side issue with which he begins and ends his remarks is the question of the factor of safety in flat slab construction, and the load tests he has made on his own constructions, showing a factor of safety of at least 4. He also "claims that flat slabs designed according to Mr. Eddy's formula have only a factor of safety of 2, under the most favorable circumstances". Now, this is not so, because, in the case of the test of the St. Paul Bread Company Building, given in the paper, and designed according to the standards of the mushroom system for a load of 100 lb. per sq. ft., one panel was loaded with 415 lb. per sq. ft., without showing signs of distress or any cracks except a few very fine hair line cracks, to be discerned only with difficulty. That observed result would seem to have more weight than any mere estimates on which Mr. Mensch bases his claims, and would tend strongly to make us put no trust whatever in the claims which he has put forth so confidently. According to him, that slab ought to have failed under half the final test load. The truth of the matter is that the materials of which it is composed are used more economically and will carry more load than if disposed as Mr. Mensch proposes in a slab-girder construction, and though he is willing to concede a slight advantage to flat slab construction, weight for weight, his adoption of a wrong theory prevents him from giving flat slabs the credit to which they are entitled.

Mr. Mensch declares that extensometer measurements of steel stresses afford no correct indication of the strength of a slab, and load tests, evidently, in his estimation, afford no reliable indication of what may be expected beyond the loads actually applied, as he believes a slab is subject to sudden collapse from sudden and unforeseen large stresses in

Mr. Eddy. the steel which may be expected to develop with slight increments of load; but, were this the case, this phenomenon should have been met frequently in the course of the numerous tests which have been made. So far from this being the case, the fact is that multiple reinforcement is of such a nature that unusual stresses developed accidentally in any rod of a slab must ultimately be distributed and divided among neighboring rods and thus be safely carried.

Mr. Mensch is mistaken in ascribing the discrepancies between observations at symmetrical points in slabs to sudden changes in stress under comparatively small increments of loading, such as have occurred in beams. A study of the stresses given in connection with the test of the St. Paul Bread Company Building, where these discrepancies were observed, fails entirely to reveal any such phenomenon as this. The discrepancies were in general as pronounced at smaller as at larger loads, with no such sudden increases of stress as alleged. This might well be expected in beams when cracks develop, but a crack such as ordinarily occurs in a slab would not have any such observable effect. It would be necessary for cracks to cross one another in the slab in a way such as they practically never do, to permit even the initiation of effects similar in character to those due to single cracks in a beam.

Almost at the beginning of his discussion Mr. Mensch turns from the discussion of slabs to that of beams, which he makes his principal side issue. He says that "one may reasonably expect that the discrepancies will be still greater in flat slab construction", because the percentage of reinforcement is less than in beams. The entire purport and effect of the reasoning and the results reached in the paper go to establish the fact that slab action and beam action are different, theoretically and numerically, and it seems almost useless to attempt to reply to criticisms based on a complete confusion of beam action with slab action. Mr. Mensch says that slab action is "simply a combination of continuous beams acting in at least two directions, which beams are connected with each other and with the columns, and the deflection in any point is common to at least two beams acting at right angles to each other." By that statement he means to say that a theory of computation based on such an assumed network of beams combined together would constitute a sufficiently close approximation in its action to that of a flat slab to afford a practical basis for obtaining the stresses in a flat slab.

The writer does not hesitate to deny that there is any such agreement between the results of such a theory and any set of trustworthy experimental observations that Mr. Mensch can produce. Mr. Mensch refers in particular to Danusso as an exponent of this correct theory, and as one who expresses real slab action in a scientific way. Danusso's papers, to which he refers, have been translated into German and published in book form, so that they are perfectly accessible to any

Mr. Eddy.

one.* As must be evident, such a network of beams necessarily has supporting girders at the edge of each panel, and the system corresponds to a slab-beam-girder construction or else to a slab-girder construction and not to a flat slab construction. Danusso, consequently, makes certain suppositions respecting the stiffness of these supporting girders, as is perfectly proper to do.

The writer has developed a slab-girder theory which is awaiting more complete experimental confirmation before publication, but any such theory must necessarily differ from flat slab theory, and it is absurd on the face of it for any one to think that the results of either theory are applicable to another construction from that contemplated in deriving it. Flat slabs without supporting girders are alone under discussion in this paper, and Mr. Mensch's definition of slab action quoted above does not apply to actual flat slab action.

Mr. Mensch says that "the advocates of many flat slab systems declare that the common beam theory does not apply to flat slabs," and that he "has never seen a similar statement in the works of St. Venant, Winkler, Grashof, Föppl, or Müller-Breslau," and that, "in particular, the case of flat slabs was solved by Winkler and Grashof by that theory". Of course, St. Venant, Winkler, and Grashof made no statement about flat slabs, for the very good reason that they had not been introduced in their day. Flat plates, however, is a different question. A flat plate is understood to be homogeneous and of the same moment of resistance throughout; a slab is neither of these, but has increased moment of resistance where required locally. St. Venant, the greatest investigator of the theory of elasticity that ever lived, was much busied with the theory of flat plates, thick and thin. A full and critical account of his researches may be found in the encyclopedic "History of the Theory of Elasticity and Strength of Materials," by Todhunter and Pearson, occupying pages 833 to 896 of Vol. I, and pages 1 to 282 of Vol. II. He did not treat flat plates on the beam theory at all, but on the basis of the exact general theories of elasticity, without regard to the approximations involved in the beam theory. He was born in 1797 and died in 1886. Grashof, born in 1826, published the second edition of his great book on "Elasticity and Strength" in 1878. This is the edition to which Mr. Mensch has referred elsewhere and is the one always quoted. It contains his investigation of homogeneous flat plates, but nothing about slabs. Those who have not ready access to this work will find the substance of the more important parts of his plate theory in Lanza's "Mechanics."

It is impossible to explain how a man of Mr. Mensch's careful scholarship could have been led to say that Grashof solved flat plates, not to say flat slabs, by the beam theory. Nothing could be farther

* "Kreuzweise bewehrte Eisenbetonplatten," Danusso-von Bronneck, Berlin, 1913, Wilhelm Ernst und Sohn.

Mr. Eddy. from the truth. Grashof is regarded as the great exponent of the slab theory, if not the originator of it, as distinguished from beam theory. It is true that Mr. Mensch in a paper of his* has used Grashof's formulas, and may have supposed that they were established as beam formulas, but such is not the case. Müller-Breslau has not treated flat plates or slabs in any of his published works. It is useless to try to have any beam theory of flat slabs accepted on the authority of any of these writers.

Danusso, in his preface, quotes at first the formula of "Grashof's plate theory", as he calls it, and later the Bach-Föppl formula, which is a convenient modification of Grashof's. It thus appears that Föppl used the plate theory rather than the beam theory that Danusso uses.

It is entirely gratuitous for Mr. Mensch to charge up any defects in the Niagara Building, to which he refers, to others than those responsible for its design, and it may be that the writer will be as ready as any one to condemn its faults, if such there be, when he is apprised of the details of the design, of which he is as yet ignorant.

Mr. Mensch criticizes the idea of saying that a set of loads extending across a slab would tend to produce a cylindrical trough or depression under it across the slab, and that another set of loads extending across the slab at right angles to the first set will also tend to produce a cylindrical hollow or trough across the slab under it, so that in the panel where these troughs cross each other the slab will be "dished" and their mutual effect will be for each trough to diminish the other. He declares that this description is "mysterious" and one that "will not enlighten any one", and yet this is the very idea of a cylindrical trough that Mr. Godfrey uses as the basis of his argument in his experiment. The idea is used by the writer merely as a semi-popular form of description, to assist the imagination in picturing this phenomenon, a phenomenon which he has investigated analytically from an entirely different point of view. It was used in order to put forth a form of explanation which would enable the reader to see how the results which had been reached by his analysis might possibly be pictured; and yet Mr. Mensch says this explanation is used because the writer does not care to express real slab action in a scientific way. It is evident that the critic has not followed the analysis, and is not even aware that that is exactly what has been done in the writer's book, where he has based his analysis on the general equations of equilibrium of the forces, shears, and moments acting on the external surface of an infinitesimal element of the slab, as is necessary in order to make a perfectly exact and rigorously scientific investigation of the slab.

It is evident that, with such divergent views of what constitutes slab action and slab theory, it would be useless for the writer to try to come to any understanding with Mr. Mensch within limits now at his

* *Proceedings*, Nat. Assoc. of Cement Users, 1911, p. 205.

disposal, especially when he insists, as he does, on the theoretical identity of flat slab construction with girder and slab construction. Mr. Eddy.

In the interest of truth and fair play there is one correction that should be made in Table 15 given by Mr. Mensch. This table, which has been quoted several times in current literature, is based on one originally given by Mr. A. B. MacMillan, in which he computed the steel in the Turner slab for two different values of unit stress, *viz.:*

(*a*) at unit stress 16 000; weight = 549 lb. per panel;
(*b*) at unit stress 13 000; weight = 718 lb. per panel.

Mr. Turner explicitly stated that his formula, $\frac{W\,L}{50}$, was based on a unit stress of 13 000 lb., and Mr. MacMillan* made the same statement. The 718 lb. is consequently the only figure for which Mr. Turner would be responsible, and the weight of 549 lb. is entirely unauthorized, as his formula of $\frac{W\,L}{50}$ avowedly was based on a unit stress in the steel of 13 000 lb., as appears by reference to his book, page 29. For an assumed unit working stress of 16 000 lb., Mr. Turner's moment formula would become, in round numbers, $\frac{W\,L}{40}$.

Mr. MacMillan further stated his ignorance respecting the amount of steel in the radial and ring rods of Turner's mushroom head, and assumed that their function was merely to support the slab rods in position at the top of the slab. The steel in the head would, in fact, be in the neighborhood of 200 lb., and it is evidently sufficient to add considerable stiffness over the columns, so that a conservative estimate of the steel per panel in Mr. Turner's construction would be 900 lb., instead of the figure wrongly stated by Mr. Mensch. This figure would be further increased by the usual laps over the column heads, so that substantial misrepresentation is made, intentional or not, by this garbled quotation. Mr. Mensch was himself present and contributed to the discussion of Mr. MacMillan's paper and has first-hand knowledge of these facts. It should be further noted that Mr. MacMillan's design, as shown by him in his paper, is almost identical with Mr. Turner's, with the omission of the inner ring rod, and his weight of steel only slightly exceeds Mr. Turner's design, as above corrected.

He states* that:

"Floors designed by his method have been low in cost, have shown remarkable powers of resisting abuse, have stood tests of twice the live load over an entire bay, not only without signs of failure, but with such trivial deflection that one is led to believe that the constituent materials are far from being stressed inordinately."

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* *Proceedings*, Nat. Assoc. Cement Users, Vol. VI, p. 266.

Mr. Eddy. Observe that this loading is not over one panel only but "over an entire bay," which disposes of Mr. Godfrey's contention that such loading would be fatal to this kind of structure.

It will be noticed that Mr. MacMillan's computation of the slab steel, according to Grashof's formula, provides nearly the same weight of steel as Mr. Turner uses, and would properly require additional steel to form a stiff head such as Messrs. Turner and MacMillan use. In round numbers, the weight of steel in either of these three slabs is, in all, 1 000 lb. per panel. These computations are really based on a flat plate theory of slab action in distinction from beam theory. The other four computed weights in the table are really based on beam theory, and require about twice as much steel, or about 2 000 lb. per panel. Beam theory practically neglects the co-action of the various belts, which co-action is considered and allowed for in slab theory. It affords a partial relief from the tensile stresses arising from the applied moments, a relief traceable to action of the bond shear of the embedment.

The statements that have frequently been made respecting the meaning of the results shown in this table seem to be misleading, as a correct interpretation of them is apparently to the effect that a beam arrangement, such as is involved for example in two-way reinforcement, requires perhaps twice as much steel as a slab which can develop true plate action.

The advocates of beam theory insist that any flat plate action that may be developed is uncertain and unreliable, and that slabs, in order to be safe, must be designed by beam theory.

The advocates of flat plate theory insist that innumerable tests have shown that flat plate action affords a perfectly reliable and dependable form of resistance when the design is properly made. The evidence for this proposition is regarded by them as perfectly conclusive, and they find it difficult, to say the least, to endure with patience the intimations which are continually made that flat slab design is essentially unsafe unless it fulfills the requirements of beam theory. It would be just as logical to insist that a dome constructed so as to resist circumferential tensions must be designed on arch theory alone, and neglect circumferential action in the dome, as to insist that a flat slab, with a multiple-way reinforcement tied together by the embedment, must neglect the effect of that embedment.

Mr. Hatt is ready freely to admit the facts of which all who have made tests on buildings are perfectly aware, namely, that the observed stresses in the steel are actually about one-half as great as would be found by considering the steel to resist the entire applied moments.

If other critics have known the facts and have not been willing to acknowledge them, they have been disingenuous; if they have not

Mr. Eddy.

known them, they were disqualified to discuss the subject by their ignorance of the essential basis of the discussion. Either alternative would seem to discount the value of the opinions which such critics have expressed, and would seem to make the basis of their opinions dubious.

When Mr. Hatt, however, refers to the tensile stresses in the concrete, which he says carry the remaining fraction of the applied forces, he evidently has in mind direct tensile stresses in concrete, acting parallel to the steel and to be added to those in the steel, such as would disappear and be ineffective whenever cracks appeared across these lines of stress. That is a naïve and quite natural explanation, and one which is evidently correct in the case of reinforced concrete beams, but can be shown to be incorrect in the case of slabs.

Owing to the stresses which take place in multiple-way reinforcement in slabs, the shearing stresses due to the bond between reinforcement and concrete play a rôle, and have a fundamental importance, that has not hitherto been realized or understood, and they produce effects such as they do not produce in beams. This has been explained elsewhere. Concrete stresses arising from bond shear may be correctly designated as indirect stresses, in distinction from those previously mentioned as directly due to bending. They are operative and to be depended on so long as the bond remains intact, which is until final failure, and long after the cracks due to the direct stresses of bending have appeared. These indirect stresses are the ones which produce the effects Mr. Hatt recognizes. His judgment tells him that direct tensile stresses are unreliable. Agreed; but should he become convinced, however, that the concrete in combination with the steel in the slab affords a resistance of a reliable kind, as it does not in beams, he would be in a position to revise his estimates as to a correct basis for safety of design.

In tests up to perhaps one-half of the design load it is usual for direct stresses in concrete to be of so much assistance to the reinforcement as to make the steel stresses comparatively small. For larger loads, up to two or three times the design load, or more, direct stresses cannot be relied on, but indirect stresses are predominant in the concrete. The true properties of reinforced concrete, as a combination which is sufficiently fine-grained to exhibit resistance such as might properly be ascribed to a single homogeneous material different from either steel or concrete, then appear. It is to this that the formulas of the book on Flat Slabs apply, as they do not to lower stresses. The material is one that is tough and not subject to sudden collapse or unexpected failure, and will show perfectly evident signs of distress long before gradually giving way. Such material must not be made

Mr. Eddy. with its bars too large or too far apart, as that makes it too coarse-grained and decreases the surface on which bond shear acts.

Comparatively few tests to destruction have been made in slabs, but those that have been made fully support the view here advanced. When, however, the experimental evidence shall become sufficient, Mr. Hatt will feel compelled, doubtless, to accept it as a fact, but the same can hardly be said of a number of those who constantly discuss slab theory in print, but who are so wedded to preconceived *a priori* conceptions as to how slabs must act as to be wholly incapacitated for any change of view, no matter what the facts may be. For reasons stated at the beginning of the paper, the writer disagrees with Mr. Hatt's opinion that the controlling factor in slab design should be the compressive stresses in the concrete rather than the tensile stresses in the steel. One reason for this is the uncertainty still existing as to the resistance which concrete or any other solid can offer to hydraulic compression. For example, does any engineer suppose for an instant that the rocks in the depths of the earth's crust, which are subject to enormous hydraulic stresses by reason of the superincumbent strata, are thereby crushed or rendered less able to resist shear or tension?

The condition of the concrete under compression in a slab is not comparable to that in a test block or cylinder in a compression test, so that the uncertainties respecting the concrete in a slab are greater, if possible, than those respecting the steel, and the measured compressive stresses in the concrete, on which Mr. Hatt would rely, furnish no reliable criterion, for it is highly probable that shearing rather than compressive stresses are the determining factors in the ultimate resistance of the concrete.

The writer does not agree with Mr. Binckley that the technical expression "flat slab" includes structures in which deep beams or girders project below the lower surface of the slab.

The mechanical action and theory of slabs combined with or supported by beams differs so much from that of true flat slabs that it is impossible to analyze them in the same way, although that has been often attempted unsuccessfully.

The writer has already perfected a mathematical investigation of beam-supported slabs such as are shown by Mr. Binckley in Fig. 21, but, owing to the fact that sufficient experimental data are not available to establish incontrovertibly the truth of his theoretical deductions, he has not felt justified in publishing his results, although they show a most gratifying agreement with the test deflections of some half a dozen buildings of this type to which he has access, and exhibit no discordant results.

The mechanical difference between a flat slab supported on beams and one supported on columns may be made evident more readily

perhaps by considering a rectangular panel than a square one. For the sake of simplicity, consider one panel of a uniform continuous plate of many panels, the plate being uniformly loaded throughout and supported at first by columns only. As an extreme case, let the length of the panel be twice as great as its width. Then the deflections at the mid-span of the long sides would be some eight times as large as at mid-span of the short sides. If it were a reinforced cantilever slab instead of a plate, however, it would be possible to increase the reinforcement lengthwise over that crosswise of the slab so as to modify greatly these relative deflections. Now, suppose that stiff horizontal beams were placed under the plate, in contact with it at mid-spans of its sides, and extending from column to column. Let these beams be raised gradually by jacks until the deflections of the sides of the plate have been entirely removed. During this process considerable mechanical readjustments will have occurred in the slab itself, and the panel load will now be transmitted by the slab to the beams instead of to the columns in such wise that the shearing stresses at the edge of the panel will be most severe at the mid-span of each side, and much more severe on the long side than on the short side, whereas originally the shears were most intense at the edges of the columns at the corners of the panel. The readjustment of shears is accompanied by a most radical change in the bending moments and fiber stresses in the plate, so much so that, whereas the stresses in the plate were originally greater lengthwise than crosswise of the panel, that is now entirely reversed. It follows that in a reinforced slab, which is designed so as to introduce more steel wherever the total tensile stresses are the greater, the design of the slab is entirely different in the two cases, and follows different principles. In fact, the solution of the fundamental differential equation of plates and slabs which applies to them when supported by stiff beams is very different from the one applicable to the case of column supports, and has not heretofore been proposed in technical literature. Mr. Eddy.

In Fig. 22 Mr. Binckley has proposed to vary the spacing of the rods in the usual diagonal and side-belts of four-way reinforcement so as to make the rods closer to one another near the lines joining the column centers and put those which are farther from these lines farther apart, and at the same time introduce such additional rods as may be necessary to make each set cover the entire area of the slab.

In the writer's opinion these additional rods constitute no improvement, but, on the contrary, are an uneconomical use of steel which would serve a better purpose if placed in the usual belts. The reason that additional rods would not seem to be an improvement is that they would not transmit the loading to the columns so directly and economically as do the rods in the ordinary four-way design. It is a well-known fact that two-way designs are necessarily somewhat

Mr. Eddy. heavier than four-way designs, and these additional rods act mostly on the same principle as do the rods in two-way designs.

The approximate equality of the maximum unit stresses in the ordinary four-way design is proof positive of the economy of the arrangement of steel in it, hence any additional steel needed should be added along the same lines, unless it can be shown that the existing equal distribution of stresses will not be thereby disturbed.

As to Mr. Binckley's proposal to decrease the spacing of the rods near the center lines of the belts: In a four-way design, with all belts of approximately equal cross-section, the unit stresses are slightly less at the center of the panel than in the side-belts. If this design is to be adhered to, the rods at the panel center are amply sufficient as they are, and there would be no economy in a change of spacing. It might be possible to make a slight reduction in the number of diagonal rods by adopting Mr. Binckley's suggestion.

In the side-belts the stresses in the several rods are approximately equal at present. There is no object, therefore, in varying the spacing.

Mr. Binckley's remarks respecting the converging compressive stresses in the concrete at the center of the panel, are pertinent, but he omits to say that just such converging compressive stresses occur around each column at the edge of the capital; there, however, they are far more severe than at the center of the panel.

The question as to how large a stress of this kind is permissible and safe is one of the most pressing and serious problems of flat slab construction. Apparently, no one knows whether the proper basis on which to decide the question is whether actual stress per square inch, the percentage of linear deformation, or the shearing deformation, affords the true criterion of the resistance of the concrete here.

As has been explained elsewhere, the writer's views of Poisson's ratio disagree completely from those of Mr. Binckley. The term "weakest section" seems to be somewhat misapprehended by Mr. Binckley and others who have treated this subject. As shown in the writer's book on "Flat Slabs", the expression refers to the ultimate behavior of the slab when loaded beyond the yield point to a point of incipient failure. The properties of the slab are then different from those exhibited under ordinary elastic or semi-elastic flexure. The actual design of such a structure should be made, not with reference to behavior beyond the yield point, but with reference to stresses during flexure, before it reaches that point; and the near equality of unit stresses at the middle of the several belts, shown by tests in well-designed slabs, shows that they are in fact now so designed.

It is often said that a true flat slab is designed so as to imitate the action of a flat plate, but that is a very imperfect statement, if it refers to a uniform flat slab. It is an imitation of a flat slab of greatly varying cross-section. No one would say that a truss bridge

is made in imitation of a beam, because the truss which varies from point to point, is far more economically designed than a uniform beam. The truss bridge may be a simple truss, or a cantilever, etc., and is differently designed in each case; it is so with a flat slab. It has a special design, and should have a reinforcement to match the design. Theoretically, it is possible to make cantilever flat slabs which are merely cantilever slabs, with negative bending at every point, or they may be made with positive moments at all points. The design determines where the lines of inflection shall be, that is, the lines which separate positive from negative bending moments. These lines must necessarily lie near those *loci* where the reinforcing steel passes from top to bottom of the slab, and the introduction of belts, some of the rods of which bend downward at one place and others at another, is in effect an attempt to prevent the lines of inflection from having any fixed positions. It is far more economical of steel to have definite lines of inflection fixed by its arrangement, and not have to provide for indefinite or undetermined stresses such as are introduced by the designs just mentioned. Mr. Eddy.

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AN INVESTIGATION OF SAND-CLAY MIXTURES FOR ROAD SURFACING.*

By John C. Koch, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. E. W. James, Arthur H. Blanchard, Spencer J. Stewart, James Owen, and John C. Koch.

Introduction.

The writer has been investigating the subject of sand-clay road construction for the past 2 years, for the purpose of establishing a sound basis for the selection of suitable materials for use in this cheap type of road. The materials are so cheap and distributed so abundantly that the writer is convinced that, if the Engineering Profession realized more fully the many advantages of such roads, there would be a much wider adoption of this type of construction.

The purpose of this paper is to set forth the results of the writer's study of these materials, both in the field and in the laboratory. This work was done under the auspices of the University of Georgia, a Good Roads Department having been established in the fall of 1911 for the purpose of carrying out a policy of University Extension work by offering free engineering assistance to the road officials of the State on highway and bridge work. The writer makes grateful acknowledgment of the facilities placed at his disposal by Chancellor

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D. C. Barrow, of the University of Georgia, and C. M. Strahan, M. Am. Soc. C. E., for many helpful suggestions.

The growing demand for improved country roads at a low enough first cost to permit the construction of a relatively large mileage in counties of average financial resources has led, in many scattered sections of the United States, to an extensive adoption of the sand-clay type of construction.

Sand and clay are the most widely distributed materials in Nature, and, in suitable mixtures, yield a road-surfacing material which can be secured at low cost, because so readily accessible, and when properly built of intelligently selected material, it will render excellent service. Briefly stated, the advantages of sand-clay mixtures for road surfacing are:

(1) Low first cost,
(2) Low maintenance,
(3) No expensive machinery for construction or repairs,
(4) No skilled labor required.

Mixtures of sand and clay in proper proportions for road surfacing often occur in Nature, though good results can be secured by making artificial mixtures. The name "topsoil" has been widely used in Georgia and other parts of the South to indicate a natural mixture of sand and clay found on the surface of the ground over wide areas in the Appalachian region. Such natural mixtures, doubtless, are to be found in many other parts of the United States.

The principal advantages in using natural mixtures of the proper proportions, as compared with artificial ones are:

(1) The expense of mixing is eliminated,
(2) The natural mixtures become compacted in much less time,
(3) Repairs are more easily made, and new materials unite more quickly to the old.

The firmness of the wet sand road and the smoothness of the dry clay road are combined in the sand-clay type, and it remains of almost uniform smoothness throughout the year. It has been known for years in many localities that there were stretches of earth road that always kept hard and smooth, shed the water, and scarcely ever needed any repairs. Fig. 2 illustrates a section of natural sand-clay road near Center, Ga., which has been in use for 60 years. In such places

the surface soil has been found to contain the proper proportions of sand and clay, which, remaining undisturbed, has become compacted into a hard, water-proof surface.

There are many miles of sand-clay roads in Georgia which have cost less than $500 per mile for the surfacing in place. They carry heavy country traffic, and the repairs, during a period of 5 years, have not averaged $5 per mile per year. Considering the fact that the first cost of construction of such roads is less than the annual interest and maintenance charges on almost every other type which has been offered as an improvement on the earth road, it seems inevitable that wherever sand-clay materials are available and of suitable quality, it is the best type to use for the average country road.

Objects of the Investigation.

The writer's objects were to determine:

(1) The proper proportions of sand and clay in sand-clay mixtures to yield the best results, namely, durability in all weathers, and absence of mud and dust;

(2) The limits between which the proportions of the two materials could vary without affecting seriously the quality of the resulting road surface;

(3) The effect of varying proportions of different sizes of sand in such mixtures;

(4) An approximate field method for the examination of soils, to determine their suitability for road surfacing.

Basis of the Investigation.

It will be readily seen that, to carry out an investigation along these lines, using artificially proportioned mixtures on short lengths of experimental road, would not only involve large expense, but probably would not be conclusive because of the difficulty of making artificial mixtures according to given proportions and maintaining such proportions. Then again, materials tracked on the experimental sections by the traffic would tend to affect greatly the value of such sections. For these reasons it was determined to base these studies on roads already constructed in various parts of Georgia, and in service from 1 to 5 years. In the course of this work the writer has traveled over 2 500 miles of roads, selecting various sections of sand-clay (including

"topsoil") roads which had been in use for some time and had worn well, and concerning which definite information could be secured as to materials used, methods of construction, and behavior of the road in all seasons of the year. From the best sections selected, samples of the road surfacing in actual service were chopped out with a hatchet for a depth of 4 in. in the wheel-tracks. Usually, a 5 by 5 by 4-in. sample was taken for laboratory study.

Samples were also taken from sections where such roads had not given entire satisfaction, on account of dust and mud. The writer has examined more than 900 samples, representing 48 counties, and comprising every typical sand-clay section of the State.

Properties of the Materials.

Clay.—The properties of a clay depend on the kaolin minerals and the impurities it contains. The kaolin particles may consist of prismatic crystals, thin plates or scales, or both. The effect of the impurities depends on the quantities of each and their fineness. Clays have no characteristic color; the pure are white, but they commonly occur as blue, gray, red, yellow, green, and purple, due to impurities.

All clays are derived from the weathering of feldspar, and the particles vary in size from small pebbles to such a minuteness as to defy measurement by the most refined methods. The finer particles readily go into solution in water, and even the larger ones are easily moved by running water. When removed in this manner and deposited at a distance, the deposits formed are called sedimentary clays. Those formed in place by the weathering of feldspar are called residual clays. In general, the sedimentary are finer grained and more plastic than the residual clays. The writer's experience has been that the latter make a harder and tougher mixture for road work than the sedimentary clays when combined with sand.

Clays vary greatly in density, plasticity, and size of grain. The important characteristics of this material for sand-clay construction are plasticity, shrinkage, and the property of "slaking". The plasticity is indicated by the ease with which a clay when wet to a certain extent can be moulded into various shapes which will be retained after the material has dried. The drying out of clay produces a shrinkage which seems to consolidate the particles into a compact mass. The extent of the shrinkage will depend on the fineness of the clay particles.

The coarser clays and those with much impurity (sand, etc.) shrink but little. The purer clays show a tensile strength of from 50 to 200 lb. per sq. in. when thoroughly dried; the coarser and impure ones have very little tensile strength. The most plastic clays resist water for a long time, but the others often crumble to pieces like quicklime. This rapid breaking down is due to the absorption of water by the pores of the clay, sometimes so quickly as to act with almost explosive suddenness. It is evident that a slaking clay would tend to break down rapidly into mud when exposed to rain and the puddling effect of traffic.

Sand.—Sand consists of particles of various sizes formed by the disintegration of the many kinds of rock in which quartz appears as an important component. As ordinarily found, the individual particles possess great crushing strength, as well as resistance to abrasion. These two qualities are of the greatest importance in the sand-clay combination.

Silt and Organic Matter.—In most of the natural sand-clay mixtures occurring in proper proportions for use directly as a surfacing material, the quantity of silt is usually from 3 to 8 per cent. The quantity of organic matter varies considerably, but, as it is light and easily washed out of the road by rains, its only effect seems to be to hold a certain quantity of moisture beneath the surface. Silt, if not in too large a quantity, seems to improve the road. In a few cases the writer has found silt and organic matter together in quantities as great as 20% of the total sand-clay mixture, and yet the road was good. Probably in such cases a large quantity of water is held by this porous material and this aids in binding the sand particles together. Thus it appears that these materials are not necessarily harmful to the mixture.

Mixtures of Sand and Clay.

Whether natural or artificial mixtures are used, the properties of both materials which are best adapted to resist the weather and the wearing effect of traffic are utilized, and the unfavorable qualities of both are largely eliminated by using the two in proper combination. It has been shown that clay in the pure state shrinks as much as 10% when dried, and on being wet again will expand to an equal extent. By mixing clay with the proper quantity of sand, these contractions and expansions may be eliminated almost entirely. There-

fore, in a sand-clay mixture for road work, the sand, being held together by the clay, which usually more than fills the voids in it, takes the wear of the traffic in dry weather. In wet weather the water quickly drains off this almost impervious mixture, and the sand resists the cutting action of the traffic, although the clay may become puddled to a certain extent.

The writer is aware of the difficulties and intricacies introduced into the subject by the wide variety of clays available and their different characteristics. Nevertheless, he believes that the methods given in this paper, which have thus far proved highly satisfactory in the selection of sand-clay materials in many parts of Georgia, will be of general value wherever such materials are to be found. This basis of study of these materials has been of great service in choosing the best of a number of equally accessible deposits of natural sand-clay mixtures. It may with equal facility be used for determining in just what proportions the different materials should be used in order to get the best mixture possible out of the available local deposits where it is necessary to make artificial mixtures. Most clays contain varying proportions of sand, even when they appear to be almost pure. Clays are often sent in from the southern part of Georgia (where the surface soil is very sandy and it is desirable to lay clay on top of it in order to mix with it and form a sand-clay surface), and on analysis are found to contain as much as 60% of sand.

Laboratory Procedure.

Equipment.—So much that is useless is often purchased for laboratory equipment that the following list is given for the benefit of those who seek definite information on this subject. These have been found to give quite satisfactory service, and need not cost more than about $50.

One metric balance, 111-gramme capacity, reading to 0.01 gramme.

One metric scale, 2 500-gramme capacity, reading to 5 grammes.

One set of sand sieves, nesting, 8 in. in diameter, Nos. 10, 20, 40, 60, 80, and 100, with dust pan and lid, all brass.

Two steel moulds, brass lined, 1 in. in diameter, 4 in. long, with 5-in. close-fitting plunger.

One wooden mallet.

Six 300-cu. cm. evaporating dishes, porcelain.

Two Bunsen burners.

Several small tripods for burners.

Several iron wire mats, 4 by 4 in., for Bunsen flame.

Thermometer, 0° to 150° cent.

Six, 2-quart, enameled milk pans for baths, etc.

The evaporating dishes should be Royal Berlin porcelain, as this is the only kind that the writer has found satisfactory.

Separation of Materials.—Samples are usually taken in several parts, so as to enable one to determine to what depth the material may be used. In some cases a sample may be in three or more parts, being 4-in. layers taken in succession from the same hole, each being kept separate from the others. After analyzing each part of a sample separately, the composition of the top 4, 8, 12, or 16 in. can be determined easily. This gives a clear idea of the value of the material and of the greatest depth to which it may be excavated.

The essential objects of the laboratory examination are to ascertain:

(1) Relative proportions of sand and clay,
(2) Physical analysis of sand,
(3) Physical properties of the mixture of sand and clay,
(4) Presence of other material which might prove detrimental,
(5) Physical properties of the clay.

To attain these objects, the following steps are necessary, in the order of their importance, and to reduce the laboratory work to a minimum:

(1) Separation of sand and clay,
(2) Mechanical analysis of the sand content,
(3) Slaking test on cylinder of sand-clay,
(4) Examination for mica and feldspar,
(5) Slaking test on clay cylinder.

1.—Separation of Sand and Clay.—The sample to be examined is first dried thoroughly in the air; then it is screened through a No. 10 sand sieve (10 meshes per linear inch), lumps and clods being pulverized with a wooden mallet beforehand. All coarse materials caught on the No. 10 sieve are considered arbitrarily as gravel, and all smaller materials are taken as the sand-clay portion. Pieces of grass, roots, etc., are discarded, of course, before weighing the two parts of the

screened sample. The material held on the No. 10 sieve is regarded as being much the same as the broken stone in a concrete mixture, and the sand-clay as the cement and sand portion constituting the mortar in such a mixture.

The part of the sample passing the No. 10 sieve is then examined as follows: By successive quartering, about 150 grammes are taken and dried in the air bath at 100° cent. to constant weight. This may often be dispensed with, and the sample merely air dried. Then 100 grammes of this material are weighed and placed in a porcelain evaporating dish; water is added and the soil rubbed well until the clay particles are in suspension. The clay in suspension is then carefully poured out and more water is added; the process is repeated until there is faint or no coloration of the water when the residue is stirred up. By washing the materials properly, practically all the organic matter, silt, and clay are removed, and only the sand particles are left, with possibly some mica and feldspar.

2.—Mechanical Analysis of the Sand.—The moisture in the residue in the evaporating dish is then evaporated, leaving the sand dry and ready to be screened. After weighing the sand it is screened through a nest of five sieves having 20, 40, 60, 80, and 100 meshes per linear inch. The weights of sand caught on each of these sieves are determined, and a curve is plotted showing the weights as ordinates and the sieve sizes as abscissas, based on 100% for the total weight of the mixture of sand and clay. Thus the weights in grammes show as percentages of the total. Fig. 1 was prepared in this way, and shows graphically the sand analyses of a number of samples.

3.—Soil Slaking Test.—A test cylinder, 1 in. in diameter and 3 in. long, is made of the material passing the No. 10 sieve by wetting it sufficiently to make a very stiff paste, and, after working it thoroughly together, placing it in a metal mould, using a tight-fitting plunger to compact the material as much as possible by tamping with a mallet. This cylinder should be dried to constant weight in the air bath at 100° cent. When entirely cooled it should be immersed completely in a glass jar of water at a temperature of 21° cent., and the time noted in which it disintegrates completely. Disintegration is assumed to be complete when the cylinder has broken down until the material is standing approximately at its natural slope of repose.

In sand-clay mixtures which have given satisfactory service, the

time to disintegrate completely may vary from 2 min. to nearly 1 hour; usually, it will be from 5 to 20 min. This test will give a fairly good idea of the resistance of any sand-clay mixture to the action of water, and, for purposes of comparison, is made more easily and

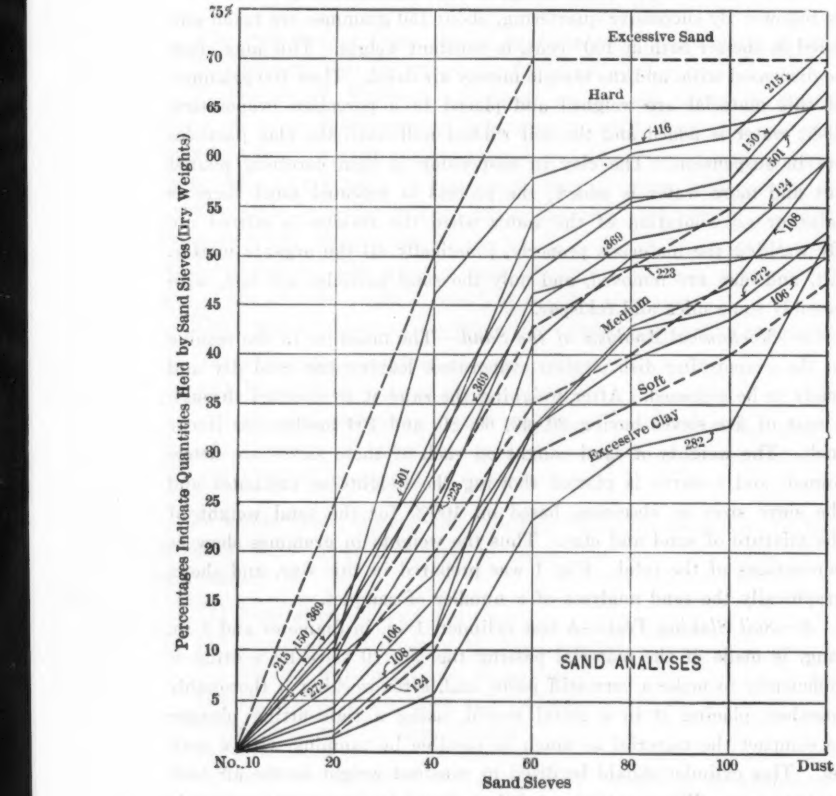

FIG. 1.

quickly than that to be described for the slaking of clay alone. The most durable mixtures, in general, are those which take the longest time to disintegrate. Mixtures containing a coarse, flaky clay may hold their shape for several days before breaking down completely, but this is not a frequent occurrence.

4.—Tests for Mica and Feldspar.—Mica is easily recognized, and its presence need occasion no alarm unless it exceeds 5% of the total sand-clay sample. This is especially true of the Southern Appalachian region, where mica is often associated with the coarser sands, as in the weathered gneisses of the crystalline areas of that region. In excess of about 5%, mica will cause considerable trouble, acting as a lubricant in wet weather and crushing into a fine powder when dry. It is easily detected with the low power of a compound microscope, a magnification of from 30 to 50 diameters being large enough, in most cases, to make out the characteristic laminated structure.

The separation of the mica is often a difficult operation. This is especially true of soils in which it is present in a finely divided state. In washing a sample of such a soil, the mica will go into suspension in the water with the clay and silt. If the clay and silt precipitate before the mica, it is easy to remove the latter by drawing off the wash-water and passing it through a paper filter. Otherwise, it would be impossible to effect a separation of these materials. Soils in which there is much finely-divided mica will usually present a characteristic, lustrous surface.

In most cases, however, the mica can be removed from the sample by washing out the clay and silt carefully (as previously described under "Separation of Sand and Clay"), the residue being sand and mica particles. A separation of these can be readily effected, the difference in the specific gravities being sufficient to permit of a gravity separation when covered with water and gently shaken. The sand sinks to the bottom and the overlying mica can be removed with a spatula. After drying, the mica is weighed and the percentage of the total sample determined.

The most harmful of the feldspathic materials, according to the writer's experience, occurs in the form of small pebbles, or even in large irregular masses, of an earthy appearance, and of a color ranging from light yellow to dark brown or black. Though easily crushed, it will generally have more coherence than the clods of earth or clay in the sample. Therefore, the larger pebbles can be separated by screening the dried sample through the No. 10 sieve. If pebbles of other material are also found, the difference in color will permit of a separation. The feldspathic materials passing the No. 10 sieve, together with the sand, clay and silt, will be left with the sand, as a

residue after washing out the other materials. Separation of the feldspar can then be made in the manner described for mica. If this material occurs in greater proportions than about 8% of the total sample, by dry weight, the road surface will cut and wash easily. This is a figure based on actual experience with a few roads, and is probably subject to modification, depending on the characteristics of the other components of any given sample. The presence of fine feldspathic material will be readily apparent in the soil cylinder test. When the soil cylinder is immersed the feldspar will almost immediately go into solution or suspension, and in a minute or so the water will become so discolored that the cylinder cannot be distinguished.

The number of cases in which a separation of either of these materials is advisable will vary considerably. In the writer's experience, about 6% of all samples examined were analyzed quantitatively for mica or feldspar. Although the methods described are approximate, they have given very satisfactory results.

5.—Slaking Test on Clay Cylinder.—The clay from the 100-gramme sample can be collected by saving the wash-water and allowing the clay to settle. Usually, the quantity of clay from such a small sample is not sufficient to make the test cylinder, so that additional clay must be removed from the material passing the No. 10 sieve to make up the cylinder. This cylinder is made by mixing the dried clay with only sufficient water to make a very stiff paste, which is then moulded under sufficient pressure to make a compact mass. After drying in air and then to constant weight in the air bath at 100° cent., the specimen is kept at that temperature until at least an hour after the last trace of moisture is found by condensation on a glass plate above the cylinder. It is important that the cylinder be thoroughly dried. When properly dried it is completely immersed in a glass vessel of proper size containing water at 21° cent. The character of the slaking and the time required for complete disintegration depend somewhat on the temperature of the water, and the freedom from moisture of the dried cylinder, so that, for a comparison of results, all specimens should be thoroughly dried and the water for the test brought to the same temperature in each case.

The slaking of clay is similar to that of quicklime, except that the action is entirely physical and not chemical. The time and character

of the slaking give an idea of the value of the clay as a binder in the sand-clay mixture. In clays which have been separated from samples taken from road surfaces which have proved satisfactory, the time for the clay cylinder to disintegrate completely varies from 2 to 20 min., the average being about 4 min. The degree of disintegration of the clay may vary from finely divided particles to flakes almost ½ in. long. In general, the coarser the particles the better the indication for suitability for road work. Clays which disintegrate completely in less than 2 min. may be regarded with some suspicion, but need not necessarily be rejected entirely, unless the sand analysis and the soil cylinder also give poor tests.

Typical Examples, with Analyses and Road Histories.—It is thought that a few characteristic cases from actual practice may be of interest. In Table 1 is given a list of the samples, with the locality from which each was taken. Table 2 shows the sand and clay analysis, based on the dry weight of the sample, the total weight of each sample being taken as 100%; all sand percentages are based on this.

TABLE 1.—Typical Sand-Clay Analyses.

Percentages of Total Weight of Sand and Clay.

Sample No.	106	108	124	150	215	223	272	369	416	501	282
Sand No. 10	(12.5)	....	....	(6.3)	....	(34.0)	....	(7.2)	(1.5)	(10.0)	....
" No. 20	4.5	3.6	2.0	13.7	9.2	6.8	9.2	12.0	10.6	11.0	1.4
" No. 40	10.8	7.6	9.0	16.6	19.5	15.9	8.6	15.7	35.4	21.4	4.3
" No. 60	17.5	21.5	30.0	17.7	25.2	23.5	14.5	17.3	11.9	17.8	17.5
" No. 80	6.7	10.2	9.5	7.5	6.5	2.8	10.3	6.8	3.3	5.8	7.3
" No. 100	4.0	4.0	1.0	2.3	1.6	0.4	2.0	2.0	2.1	0.4	2.1
" Dust	6.5	13.0	8.5	11.5	9.5	2.1	8.0	10.0	2.0	10.4	19.1
Total Sand	50.0	59.9	60.0	69.3	71.5	51.5	52.6	63.8	65.3	66.8	51.7
Clay	50.0	40.1	40.0	30.7	28.5	48.5	47.4	37.2	34.7	33.2	48.3
Sand 20-60	32.8	32.7	41.0	48.0	53.9	46.2	32.3	45.0	57.9	50.2	23.2

NOTE.—Sand No. 10 is based on gross sample = 100%, and includes all gravel. Remaining portion of table is an analysis of material smaller than No. 10.

Fig. 1 shows a graphic analysis of these samples, with the sand analysis in detail. This diagram also gives, graphically, the limits between which the writer classifies his samples, basing such classification on the sand analysis as well as the clay content, both of which are shown.

The limiting curves separating the analyses into four groups are based on the writer's experience, and are given with full confidence in their utility. They are of sufficient accuracy to be of considerable

service in classifying sand-clay mixtures properly and quickly, and arriving at a reasonable approximation of their value as road material. Quite often it will be found that the sand analysis alone may show that the material is unsuitable, and much time may be saved that might otherwise have been lost in continuing the examination and applying other tests.

TABLE 2.—ROAD HISTORIES.

Sample Numbers Refer to Corresponding Numbers in Table 1.

Sample No.	County.	Road.	Sample taken.
106	Bulloch	Statesboro-Savannah	6.7 miles south of Statesboro.
108	Dougherty	Albany-Thomasville	4.3 miles south of Albany.
124	Sumter	Americus-Albany	6.0 miles south of Americus.
150	Clarke	Athens-Danielsville	3.2 miles northeast of Athens.
215	Dougherty	Albany-Thomasville	2.1 miles south of Albany.
223	Richmond	"Augusta gravel" from pit.	......
272	Habersham	Clarkesville-Cornelia	1.2 miles from Clarkesville.
369	Clarke	Athens-Barnet Shoals	4.3 miles from Athens.
416	Hall	Thompson's Bridge	2.0 miles north of Gainesville.
501	Clarke	Athens-Whitehall	2.0 miles from Athens.
282	Decatur	Bainbridge-Jacksonville	7.0 miles south of Bainbridge.

The analyses falling in the group marked "Hard" will usually give a durable, hard road surface which wears exceedingly well, and, after consolidation, can be cut only with great difficulty by a road machine, if at all. Analyses in the "Medium" group will give an excellent, smooth, hard, surface, but one which may cut a little in protracted wet weather. Such a surface is too hard to attempt to shape up with a road machine, except after very long wet periods. This may be taken as a sort of "average" quality of sand-clay.

The analyses grouped in the "Soft" class give a surface which is much superior to the ordinary earth road, but which will tend to be somewhat dusty in dry weather, and will cut easily and tend to wash in wet weather. Material of this type requires re-shaping, after every heavy rain, with a road machine or drag. The class "Very soft" comprises all materials which have too large a clay content to give a satisfactory road surface; they should not be used at all, as the expense is scarcely warranted for the kind of road surface that can be obtained with them.

The writer's method is to make the separation of sand and clay, and then the mechanical analysis of the sand. If the material shows up poorly in the sand analysis, the further tests are not made, unless,

after making a similar examination of all other available materials, it appears to be the best available. Each case is a special problem in economy, and a study of each will indicate the proper method of using local materials if they can be used at all.

Road Histories.

The following is a brief history of the various roads, samples of which were analyzed and the results presented in Table 1, and graphically in Fig. 1:

106.—Bulloch County.—Had been in service 3 years at time sample was taken. Excellent, hard, smooth surface, but occasionally dusty in prolonged dry seasons. Softens in wet weather and cuts badly then. Re-shaped after heavy rains with light drags. Artificial mixture.

108.—Dougherty County.—Good smooth surface in dry weather. Road in service about 3 years at time sample was taken. Softens and washes in heavy rains. This material, found about 2 ft. below natural surface, is used without addition of other material.

124.—Sumter County.—Very good, smooth surface, but softens and washes badly in wet weather. Re-shaped with light drags after even moderate rains. This is generally true of all the roads in this county, the materials available being very similar in mechanical analyses. Natural mixture found 2 ft. below surface.

150.—Clarke County.—Road in use 4 years, with practically none but the very lightest repairs, when sample was taken. Cost of maintenance, as given by superintendent, averaged less than $5 per mile per year. Very little dust; firm, hard road surface throughout the year, though freezing and thawing might at times soften the crust for a depth of an inch or so. No re-shaping with road machine possible, on account of the hardness of the compacted material. This seems to represent the best and most durable type of sand-clay combination yet found in Georgia. Natural mixture found as topsoil of cultivated fields.

215.—Dougherty County.—Road in use about 3 years at time sample was taken. Good hard surface, softens but little in wet weather, and wears well, keeping good shape. Too hard to re-shape by dragging. Few repairs. Natural topsoil.

223.—Richmond County.—Reports of roads on which this material had been used indicate that it produces very satisfactory results. This

material is shipped by rail for road construction for distances as great as 100 miles. The analysis in Table 1 indicates that this is really a low-grade gravel rather than a sand-clay mixture. Yet it is interesting to note that the mechanical analysis of the portion of this material smaller than No. 10 mesh agrees very well with the mechanical analysis of the better classes of sand-clay mixtures which, with very small gravel contents, have given results almost as good. The mechanical analysis is shown on Fig. 1, considering only the portion smaller than No. 10 mesh. This material is dug from a large pit near Augusta, and has been in use for many years.

272.—Habersham County.—The road from which the sample was taken had been in use for about 8 months. The surface is hard and smooth, and softens but little in wet weather. This material was selected by the writer after sampling a large number of local deposits. Haul, about 2 000 ft. Cost in place, for 16-ft. width of surfaced roadway, about $500 per mile. Local officials, after using macadam at about $6 000 per mile, pronounce this a better road than macadam, even if initial cost and maintenance were equal. Local natural mixture of sand-clay from topsoil of cultivated field.

369.—Clarke County.—Built about 3 years at time sample was taken. Hard, durable surface. Few repairs. Too hard to drag. Experience quite similar to that of No. 150. Natural topsoil sand-clay mixture, obtained from nearby cultivated fields.

416.—Hall County.—In use less than a year. Replaced macadam road that cost $6 000 per mile, and gives better satisfaction than the macadam. Cost $350 per mile for 16-ft. width of roadway. Material natural topsoil from nearby cultivated field.

501.—Clarke County.—Road from which sample was taken had been in use about 3 years. This road was surfaced with a very thin layer of natural topsoil from adjoining fields, so that, after being compacted by traffic, its thickness was about 3 in. In seasons of prolonged wet weather traffic occasionally cuts through and mud-holes are formed. The material is excellent, but has not been used in sufficient quantity to give the best results. A comparison of its mechanical analysis with that of other samples indicates its high quality.

282.—Decatur County.—The road from which this sample was taken had been in use about one year. This is practically no improvement over the ordinary earth road, as the sand content is so low as

FIG. 2.—NATURAL SAND-CLAY ("TOPSOIL") ROAD, NEAR CENTER, GA. IN USE ABOUT 60 YEARS.

FIG. 3.—NEAR THE SAME PLACE AS SHOWN BY FIG. 2. NEW ROAD ON THE RIGHT.

FIG. 4.—TYPICAL SAND-CLAY ROAD IN THOMAS COUNTY, SOUTHERN GEORGIA.

FIG. 5.—TALLASSEE ROAD, CLARKE COUNTY, GEORGIA. TOPSOIL IN PLACE SIX DAYS, WITH TWO DAYS OF HEAVY RAIN.

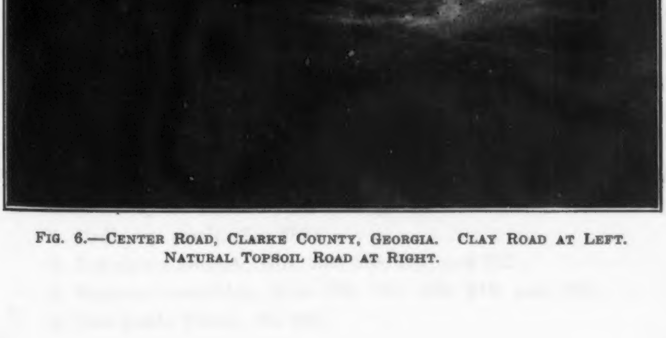

FIG. 6.—CENTER ROAD, CLARKE COUNTY, GEORGIA. CLAY ROAD AT LEFT. NATURAL TOPSOIL ROAD AT RIGHT.

[illegible]

[illegible]

to be of no value in improving the quality of the road surface. It is very dusty in dry and very muddy in wet weather, after the traffic has been on it a few days. As difficult to maintain as an ordinary earth or clay road. Material used was taken from adjoining fields at a depth of 2 ft.

Table 3 is a summary of the road histories and the corresponding analyses, for comparison. For simplicity, the portion of the sample larger than No. 10 will be considered apart from that smaller than No. 10.

Effect of No. 10 Sand.—Comparing Sample No. 106 with the others, it is found to contain a larger quantity of No. 10 sand than any of them, with the exception of No. 223. No. 223 is really a low-grade gravel, and is discussed in a succeeding paragraph. The history of No. 106 is not nearly so satisfactory as that of the remaining samples, except Nos. 108 and 282. Sample No. 108 is slightly more satisfactory than No. 106, and No. 282 is practically an earth road. Furthermore, Samples Nos. 124, 215, and 272, which contain no No. 10 sand at all, have given much more satisfactory service than No. 106.

It has been found by the writer, in samples of gravel and chert roads (Augusta gravel being a typical material), that the part larger than No. 10 varied considerably in the few samples examined, although the composition of the part smaller than No. 10 was fairly constant in the proportion of sand to clay. The gravel part of any sample may reasonably be taken as an approximate measure of the durability of a road built with such mixtures. In addition, if a sample has a considerable proportion of gravel and is deficient in sand, the presence of the gravel will largely offset such defect.

From the foregoing it is clear that the effect of No. 10 sand on a mixture may be, either that of making up a deficiency of sand in the portion of the sample smaller than No. 10, or to increase durability when there is no deficiency of sand. Hence, its effect depends largely on the analysis of the portion of the sample smaller than No. 10.

Material Smaller than No. 10.—Based on their histories, the samples of Table 3 may be divided into four groups:

1. Ordinary earth, No. 282;
2. Inferior sand-clay, Nos. 106, 108, 124, and 272;
3. Superior sand-clay, Nos. 150, 215, 369, 416, and 501;
4. Low-grade gravel, No. 223.

TABLE 3.—SUMMARY OF ROAD HISTORIES AND ANALYSES.

Sample No.	106	108	124	150	215	223*	272*	369	416*	501	282*
Character of Road Service:											
In dry weather	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Medium
In wet "	Soft	Soft	Soft	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Soft
In prolonged weather	Cuts 2"-6"	Cuts 2"-4"	Cuts 1"-4"	Cuts ½"-1"	Hard	Hard	Cuts 1"-2"	Cuts ½"	Cuts ½"	Cuts ½"	Cuts 6"-10"
In freezing weather	Cuts 2"-6"	Cuts 2"-4"	Cuts 1"-4"	Cuts ½"-1"	Cuts ½"-1"	Cuts ½"-1"	....	Cuts ½"	Cuts ½"	Cuts ½"	Cuts 6"-10"
Frequency of re-shaping: Times per year	10-20	10-20	15-25	None in 4 yrs.	None in 3 yrs.	None in 4 yrs.	None in 8 mos.	None in 3 yrs.	None in 7 mos.	None in 3 yrs.	2-4
Sand No. 10, percentage	12.5	....	....	6.3	....	34.0	....	7.2	1.5	10.0	....
Sands Nos. 20-60, percentage	32.8	32.7	41.0	48.0	53.9	46.2	32.3	45.0	57.9	50.2	23.2
Total sand, percentage	50.0	59.9	60.0	69.3	71.5	51.5	52.6	63.8	65.3	66.8	51.7
Total clay, percentage	50.0	40.1	40.0	30.7	28.5	48.5	47.4	37.2	34.7	33.2	48.3

NOTES:—* Data for No. 223, based on roads built near Augusta, Ga.
No. 272, Road in use 8 months at time of report.
No. 416, Road in use 7 months at time of report.
No. 282, practically an earth road.

Considering only the second and third groups, the sand-clay analyses may be condensed into Table 4:

TABLE 4.—COMPARATIVE ANALYSES OF SAND-CLAY GROUPS.

	INFERIOR SAND-CLAYS.		SUPERIOR SAND-CLAYS.	
	Limits.	Average.	Limits.	Average.
Sand total	52.6-60.0%	56.3%	63.8-71.5%	67.6%
Sand Nos. 20-60	32.3-41.0%	36.6%	45.0-57.9%	51.4%
Sand No. 100 and dust	10.0-17.0%	13.5%	4.1-13.8%	8.9%
Clay total	40.0-47.4%	43.7%	28.5-37.0%	32.7%

The principal differences between the two groups are:

1. The total sand content in inferior mixtures averages about 11% less than in superior mixtures;
2. The total quantity of sand from Nos. 20 to 60, in the former averages 15% less than in the latter.

Effect of Size of Sand.—Although the quantity of clay is nearly the same in Nos. 106 and 282, the former is a much more durable material. Roads built of the former withstand severe rains far better than those built of the latter material. To account for this, it must be remembered that the analysis of the whole sample of No. 106 is: Sand No. 10, 6.5%; sand No. 20 to dust, 43.7%; clay 43.8 per cent. The difference in the clay content is only about 4% in the two samples, which seems too little to account for the great difference in their conduct. By reference to the sand analyses of the two samples, it is seen that No. 106 contains nearly three times as much of the No. 20 and No. 40 sands, nearly the same quantities of Nos. 60, 80, and 100 sand and only one-third as much dust, as No. 282. This considerable difference in the ratio of coarse to fine sands explains, in large measure, the difference in the durabilities of the two materials.

CONCLUSIONS.

A study of nearly a thousand analyses, in the laboratory, and a comparison of the results attained with such materials in actual road construction, have led the writer to formulate the following tentative working rules for examining sand-clay mixtures and as a basis for making up proper combinations for artificial mixtures:

(1) The total relative sand content, disregarding the size of the sand grains, is no criterion of the value of the material.

(2) The sand smaller than No. 60 is of little value in the mixture, that smaller than No. 100, except in very small quantities, is detrimental.

(3) The greater the proportion of coarse to fine sand the harder and more durable will the road surface be.

(4) For the best possible results with sand-clay mixtures, the sand smaller than No. 10 and larger than No. 60 should not be less than 45% nor more than 60%, by dry weight, of the entire sample. In addition, the sand smaller than No. 10 and larger than No. 60 should be composed of about equal parts of Nos. 20, 40, and 60. The total sand content should in no case exceed 70% by weight, of the total sample.

(5) Test cylinders of the sand-clay mixture, 1 in. in diameter and 3 in. long, should, when thoroughly dried in air bath at 100° cent., take at least 2 min., when immersed in water at 21° cent., to crumble down to the natural slope of the material, and preferably should take 6 min. If the cylinder fails in this test, it should be regarded with suspicion. If the sand analysis is poor and the cylinder test is also poor, the material is not worth using.

(6) Test cylinders, made from the clay removed from the sample, 1 in. in diameter and 3 in. long, should take at least 2 min. to crumble down to the natural slope of the material, when immersed in water at 21° cent. If it fails in this test, but passes the test of the preceding paragraph, it may be used, but it indicates a poor quality of binder.

Approximate Field Method for Examining Sand-Clay Mixtures.

As a rapid aid in forming a judgment in the field as to the value of any mixture, the writer offers the following: A buggy inspection is made of the natural soil on both sides of the road to be improved. Samples are taken wherever the surface appearance seems to indicate favorable material. These samples are placed in paper bags, with notes of the location, etc. The samples are usually taken for a depth of 4 in. Each sample can be dried sufficiently by spreading out in the sun in a thin layer. When dried, the sample may be placed in

a No. 10 sieve and screened, and note made of the relative weights of the residue on the screen and the finer material. Then 100 grammes of the part passing this screen are weighed and placed in an evaporating dish. By careful washing, in a few minutes practically all the clay may be washed out, and is rejected. The sand residue is then dried with an alcohol flame, and weighed. It is then placed in a No. 20 sieve nesting over a No. 60 sieve. By screening the sand thus it can be quickly separated into three sizes: No. 20, between Nos. 20 and 60, and smaller than No. 60. These three portions can be weighed, and from this information a very good idea may be obtained of the comparative value of the material, without additional tests.

The proportion of the material coarser than No. 10 indicates the proportion of gravel present. The separation into No. 20 size and between Nos. 20 and 60 shows practically into what class the mixture would fall, and the portion smaller than No. 60 would indicate the quantity of almost worthless material in the mixture. By using Fig. 1 a very good idea of the quality of the sample would be gained.

Methods of Sand-Clay Construction.

There are two general methods of construction in use, one in which the natural mixture of sand and clay is used, the other in which an artificial mixture is made by using two or more materials which are mixed by plowing together, puddling, etc.

Construction with Natural Mixtures.—The method which the writer believes is simplest, and produces excellent results, is as follows: Natural mixtures of the proper proportions of sand and clay may be found as the natural topsoil of cultivated farms, or may be found below the surface at various depths. The material comprising the topsoil of cultivated land, when composed of the proper combination of sand and clay, has probably been more thoroughly weathered and therefore is less likely to wash and disintegrate when placed on the road. Cultivation has also produced a more thorough and complete mixture of the two materials.

In the northern half of Georgia the topsoil in large areas, especially on the tops of ridges, for a depth of from 6 to 12 in., is found to be an excellent sand-clay mixture. In the southern half of Georgia, a natural mixture of sand and clay is often found at a depth of from 2 to 5 ft. below the surface. Such material is not usually very well

weathered, so that, after being placed in the road, a certain quantity is found to wash out quite readily with the first few rains.

In either case the method of construction may be the same. The sub-grade of the roadway is brought to a level or slightly convex cross-section. The sand-clay is then placed in a continuous layer, from 10 to 12 in. thick, the material being spread as fast as delivered and not dumped in piles here and there. This layer is spread for a width of 20 ft. for a nominal 30-ft. roadway. After a sufficient quantity has been placed in this manner, an ordinary road machine is drawn along the ditch line, cutting about 4 in. deep at the outside, and the blade is set so as to cast the material from the ditch against the edge of the sand-clay layer. In this way a shoulder is built up against the sand-clay to hold it in place. This also shapes the ditch. After both sides have been thus shaped, the road machine, in successive passages, rounds up the cross-section of the sand-clay so as to give proper crown to the roadway and a smooth line from the crown to the ditches. As soon as the road is shaped, traffic and the construction teams begin to compact it, and it rapidly becomes consolidated without the use of a road roller. As the consolidation progresses, ruts are formed, and they should be filled and a proper cross-section maintained by the occasional use of the road machine for a period of about 2 months. Unless this is done, the road surface will become rutted and rough, and eventually compacted with a concave crown which will prevent proper drainage. After the material has been consolidated into a hard mass, the difficulty of securing a good cross-section is largely increased.

The cross-section which seems to have given the most generally satisfactory results is a parabolic form with a crown of ½ in. per ft., that is, for a roadway surfaced for a width of 20 ft., the crown would be 5 in., and the height of the center of the road above the ditch (for a road having a width of 30 ft. between ditches) would be 7.5 in. With steeper crowns than this it has been found that the surface cuts into a series of parallel ridges running from the wheel tracks to the ditches and making it very disagreeable for travel. If less crown is given, the provision for wear is too small, and the drainage may not prove satisfactory after a comparatively short time.

For several months rains are apt to soften the top crust and cut up the smooth surface, but if patience is exercised and the road

JEFFERSON ROAD, CLARKE COUNTY, GEORGIA.

FIG. 7.—TOPSOIL PLACED SIX HOURS BEFORE PHOTOGRAPH WAS TAKEN.

FIG. 8.—TOPSOIL AFTER ONE WEEK OF TRAFFIC, WITH THREE HEAVY RAINS.

FIG. 9.—JEFFERSON ROAD AFTER SIX MONTHS OF TRAFFIC, OCTOBER, 1912, TO MARCH, 1913. TWELVE HOURS AFTER HEAVY RAIN.

machine is used to maintain the cross-section properly, it will be found that the puddling action of the traffic when the road softens is a great aid to final consolidation.

Construction with Artificial Mixtures.—From analyses made of materials proposed for use on account of accessibility, and from a study of their sand analyses, the proper ratio in which two or more materials should be mixed can be determined so as to secure the best possible results with the available materials. Three cases arise in which artificial mixtures are to be used:

(1) Sand foundation, where clay is to be hauled and proper mixture made by disk plowing and puddling.

(2) Clay foundation, where sand is to be hauled and mixture made as above.

(3) Soil foundation, where both sand and clay are to be hauled and mixture made.

In any of these three cases the proper mixture of the materials and the puddling action of traffic are necessary to secure a good consolidation. It takes considerable labor to secure a satisfactory mixture, but, except for this, there is no essential difference in the fundamental principles applying to construction with either artificial or natural sand-clay mixtures. The use of the road machine to maintain the cross-section and the height of the crown should be the same for each type of construction. For the softer varieties of sand-clay, the split-log and other forms of light drags may be used effectively in maintenance.

DISCUSSION

Mr. James. E. W. James, Assoc. M. Am. Soc. C. E. (by letter).—The author has been particularly fortunate in his studies of sand-clay mixtures for road surfacing, in that his investigations have been made at the best point in all North America for such a purpose. The region especially concerned in the sand-clay type of construction is the Coastal Plain between the Upper Piedmont and Tidewater. According to State lines, therefore, Georgia, and especially North Georgia, is in the center of the sand-clay territory. The best natural mixtures are probably found wherever the Orangeburg sands of the Lafayette formation are prevalent or common. Having confined his investigations to these conditions, the author's conclusions are doubtless stated a little more strongly than they would have been had his experience or studies covered a wider territory. It is safe to state that sand-clay roads can be built in few sections of the United States in such a way as to give as great a degree of service as in Clarke or Elbert Counties, Georgia.

For several years the writer has had a wide experience in sand-clay work throughout all the Coastal Plain from Virginia to Southern Texas, and recognizes the importance of this type of construction. For a large section of the Southeastern States it is the only possible type of improved road for a large mileage of the country highways. The importation of stone is prohibitive because of excessive cost; shell is too friable and soft; brick, even on a natural sand foundation, is too expensive; and concrete is practically out of the question because of the cost of both sand and aggregate.

For many counties in the Southern States, therefore, the sand-clay road is the only improved type within the means of the locality, and the only one economically warranted by the conditions of traffic at the present time and probably for many years to come. In many sections, especially in the Tidewater and Coast counties the natural soils are prevailingly sandy. It is difficult to find good clays. The sand beds are as bad as, or worse than, the clay mud of the uplands, and, moreover, they are bad, not only in winter, but all the year round. For this reason the sand-clay road is especially valuable in these sections. The natural soil is well drained, and the latitude precludes any lasting or great degree of frost. The question in these regions is not commonly to select materials that will produce a sand-clay of certain stability, but to use the materials at hand so skilfully as to produce alleviation of the almost intolerable conditions presented by the sandy roads.

In the counties along the coast, and usually for about 100 miles inland, the sands are fine and often floury; all clay, except occasional small pockets of white or gray pipe-clay, or a mottled red and gray

variety, carries a large percentage of this fine sand. Consequently, the field methods of testing the clays described by the author would, in the writer's experience, result in rejecting a majority of those locally available, and of discarding all but the rarest deposits of sand. Mr. James.

The author's conclusions, however, are recognized as well founded, especially in regard to the value of coarse sand.

For sand-clay mixtures, the writer has frequently used a most simple field test, which has served his purpose well. The sources of material are located and selection is made, depending usually on the length of haul and the depth of the deposits beneath the surface. The immediate problem is to determine the best mixture of the materials selected.

Typical samples are taken of both sand and clay. Mixtures are made, ranging from 1 part sand to 3 parts clay, up to 3 parts sand to 1 part clay, or sometimes beyond these limits, if the materials appear to warrant it. These mixtures should be made to vary by one-half of 1 part, 1:3, 1:2½, 1:2, etc., 1½:1, 2:1, 2½:1, etc., and should be worked up with water into putty-like masses.

From each test, mix a small sample of from 1 to 2 cu. in., cut out with a small measure. The writer has found a small medicine glass, or even a large brass thimble, handy. It is only essential to get equal samples from each test mix. These samples are then rolled between the palms of the hands into reasonably true spheres and placed in the sun to dry. Some designating marks may be scratched on them. When thoroughly baked, they are placed in a circle in a flat pan or dish, and enough water is poured in the pan to cover them, care being taken not to pour the water on the samples.

Slaking will begin at once. The lapse of time found by Mr. Koch with his compressed specimens is not found at this stage. The slaking, however, will proceed at different rates. The sandy specimens will break down first, those with excessive clay will disintegrate second in order, and those having about the proper proportions will act more slowly. Usually, there will be one or two that determine the proper proportions of the materials, and, in the writer's experience, these will usually lie together in the series of test mixtures.

A supplementary test of some value can also be made on the dry spheres. Lightly rubbed with the thumb, those having too much sand will break down rapidly. Those having too much clay will soon begin to "dust" away, and those having the most stable mixtures will assume a slightly glazed effect under the light rubbing, due to the moisture and oil of the skin. These two tests will not give the same results. The dry test will indicate a mixture richer in clay as the better one, and the wet test will indicate a sandier mixture. The sample indicated as satisfactory under the wet test that lies between the other two will prove best in service.

Mr. James. These extremely simple tests do not determine the sand or clay content of the mixture, for the clay selected almost always contains considerable sand, and this, in most cases, contains silt or clay. The tests, however, serve to fix the actual values in the mixture of the pit run, as represented by the samples.

Mr. Blanchard. ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—The speaker's discussion will be limited to calling attention to certain features of the construction of sand-clay roads which it is believed the author should include in his very valuable contribution to the literature on this subject.

The author states that the first cost averages $500 and the annual cost of maintenance is $5 per mile; and then emphasizes the fact, agreed with by the speaker, that there is a place for the sand-clay road. Very little information, however, is given with reference to local conditions. As the fundamental principle of sound practice is to use that type of road or pavement which is economical and suitable for a given set of conditions, it would be of great value to American engineers to have at hand especially more detailed knowledge relative to the traffic on the various types of roads referred to in the paper.

Another part of the paper which it is believed could be advantageously amplified is that relative to the effect of different percentages of sand-clay mixtures retained on the 10-mesh sieve, when such mixtures are used under various climatic and traffic conditions.

There are many highway engineers who, without doubt, will disagree with the author in dividing gravel and sand on the basis of material retained on or passing the 10-mesh sieve. The speaker believes that the 4-mesh sieve is rapidly being adopted as the basis of division between these materials.

Mr. Stewart. SPENCER J. STEWART, ASSOC. M. AM. SOC. C. E.—The speaker has read this paper with great interest. From it it is evident that the clay-bearing materials are being given proper economic consideration in the highway development of the South. In New York State, however, clay is a material which, according to many highway engineers, possesses little or no value, but, on the contrary, contributes characteristics highly detrimental to proper highway construction.

Specifications prepared by highway commissioners, especially those of the State of New York, have insisted that the foundation course shall be filled with screenings, gravel, or sand, and have invariably ruled against a filler which would be classified as clay. From the speaker's experience, covering responsible charge of the construction of more than 300 miles of highways, at a total cost of $4 000 000, he has been led to believe that a clay-bearing material, if applied as a filler when dry, makes a foundation as stable as a filler of sand

or the screenings of a non-cementitious stone, and less likely to disintegrate. **Mr. Stewart.**

It is common knowledge that clay may be used to advantage in binding the top course of macadam roads in cases where the dust from the crushed stone possesses no cementitious quality.

Perhaps it is not inappropriate, and may prove interesting, to call attention to the results obtained from the use of the sand-clay-bearing gravels of the Hudson River as a material for successful road construction. What is true of the cementitious gravel of the Hudson would be true of similar deposits in other parts of the country, but it should not be presumed that every gravel bank necessarily contains the characteristics of a cementitious gravel.

This material has been used for years on the Parkway Systems of Greater New York, originally as a water-bound material and, more recently, covered with hot oil. The roadways proved very satisfactory until the advent of the motor bus, which unusual traffic they could not withstand, nor was it ever intended that they should.

This same material has been used on the sandy soil of Long Island with excellent results, in spite of the crude method, or rather lack of method, used in the construction of some of the highways. Frequently, it was dumped on the sandy road, spread carelessly, and left for the traffic to compact into a smooth surface. In spite of this, however, it soon ironed out and made a most excellent road covering, considering the time and money spent on construction. On the other hand, under the direction of the New York State Highways Commission, roads of the highest type have been built of this material by the so-called water-bound method and the application of hot oil, proper care being exercised as to rolling and puddling.

With respect to roads of this latter class, the speaker wishes to call attention to one partial failure, point out the causes, and perhaps draw a lesson from the experience.

Hot oil had been applied during the late fall, and properly covered, but, in the following spring, the heat of the sun caused the oil to bleed and adhere to the iron tires of slow-moving vehicles, which drew up part of the top course and deposited it a few feet ahead. This resulted in alternate holes and mounds, and caused a very uneven surface which gradually became more pronounced as the holes became larger through wear. As soon as the first indication of bleeding occurred, the surface should have been covered with sand to prevent this unfortunate condition. One enterprising citizen did apply this remedy in front of his own property, with the result that the top surface is as good to-day as it was at the time of the completion of the road.

Hot oil treatment on water-bound roads, whether of stone or gravel, must have a certain amount of immediate attention during the period

Mr. Stewart. when the oil is susceptible to bleeding. Such attention simply means the application of blotting material.

As a foundation course on sandy soils, cementitious gravel has proved an unqualified success, at least in the speaker's experience, for he has known of no failures. Failures do not mean that a stretch of 50 ft. in 10 miles might not prove unsatisfactory; that might happen with any foundation, due to causes which could not be guarded against at the time by the construction engineer. The larger the mineral aggregate, the better the foundation, if just sufficient sand and clay are used to fill the interstices. If a mixing-method top is to be placed, the speaker would advise that the foundation course be thoroughly puddled and dried before the top course is laid.

As a foundation course for country highways, it is believed that gravel containing fines of a cementitious nature is to be preferred to either broken stone or concrete.

First, it costs less. Incidentally, it might be of interest to know that practically all materials entering into the construction of roads on Long Island must be imported, whether it be crushed stone, Hudson River gravel, or so-called Long Island gravel. The latter exists only on the north shore of the Island in any quantity sufficient to be used economically for work of any moment, must be dredged, barged to Long Island City, there transferred to cars, and then hauled to its destination. Even then, this material should only be used in concrete construction.

Where materials have to be imported, it should be remembered that broken stone, even at the same cost at the point of destination, costs approximately one-third more than gravel, due to its consolidation of about 33% under the weight of the roller, and also to the necessity of adding from 25 to 30% of sand or screenings to fill the voids. On the other hand, gravel, when measured in bulk on barges or cars, will about equal the quantity rolled in place on the road, requiring no extra material for filling. Concrete requires about 80% of the rolled quantity of stone and gravel, with the addition of 40% of sand, together with the cement, the quantity of which depends on the proportion of the mixture.

On the contrary, where the importation of road-making material is unnecessary, the use of cementitious gravel saves the cost of crushing and the incidental expenses associated with such process.

Second, a proper gravel foundation possesses the resiliency which is generally recognized as necessary for a successful road, especially for country highways. Broken-stone foundations possess this characteristic in a less degree, and concrete is entirely lacking in it.

In the case of asphalt pavements, they are now designed to overcome the lack of resiliency of the concrete foundation by the use of an intermediate or cushion course between the concrete and the asphalt

surface. In the same manner properly designed mixing-method pavements of small mineral aggregate when placed on concrete foundations, should be provided with this cushion course. Mr. Stewart.

Where conditions of traffic require a pavement of considerable permanency, the mixing method may be used. The speaker designed a pavement consisting of a mixture of asphalt and gravel in the proportion of 1 cu. yd. of loose gravel to an average of 20 gal. of asphalt, the gravel containing not less than 10% of clay. The gravel was bank-run, the largest particles of which were 2 in. in the longest dimension, containing sufficient fines to fill the voids partly. The bitumen was a fluxed natural asphalt with a penetration between 10 and 13 mm. when tested for 5 seconds at 77° Fahr., on a No. 2 needle weighing 10 grammes.

The gravel was heated in a mechanical revolving dryer with a temperature of more than 225° Fahr., after which the asphalt, heated to not less than 275° Fahr., was added; then the mixture was placed in a revolving mixer until thoroughly and completely coated with bitumen. The mixture, at not less than 225° Fahr., was spread on the prepared bottom course with shovels from dumping boards, and raked to a uniform surface with hot rakes, after which it was rolled with a self-propelled roller weighing at least 10 tons, until it was thoroughly consolidated.

On some sections of this pavement there was placed ½ in. of gravel screenings containing not less than 10% of clay which was saturated with water and rolled thoroughly and continuously until a clay mortar had been obtained. This process filled all the surface interstices with a gritty and adhesive substance which made the road practically "non-skid". In a short time the traffic drove away all surplus screenings, leaving a mosaic surface.

A different method of treatment was applied to this form of pavement in the more thickly settled communities. After the mixture had been rolled, it was covered with a coat of hot oil as a seal coat, this seal coat being applied in November. In the following spring, under the action of the sun, the road bled to some extent, and became so sticky in a few places that the oil adhered to wagon wheels, which pulled up some of the top course. This condition could have been avoided if the authorities in charge had covered the pavement with sand or gravel screenings as soon as it became apparent that the hot oil had a tendency to bleed. No material harm was done, however, as the continuous traffic carried sufficient sand and dirt on the pavement so that in a short time the stickiness of the oil had disappeared.

Such treatment obviates the general complaint against the so-called stone-mixing method pavement where a greater quantity of asphalt is used and where the seal coat is of the same consistency as the asphalt binder in the top course proper. These objections arise from the

Mr. Stewart. hardness and slipperiness of the surface, which, during the greater portion of the year, make it undesirable for horse traffic because of the former characteristic and to motor traffic on account of the latter.

It is suggested that, where the hot oil is omitted at the time of the original construction, the mosaic surface be covered with a hot oil treatment of ½ gal. per sq. yd. during the following year. With the hot oil as a squeegee course, in place of the seal coat of the same consistency of the bitumen, the pavement proper gives a less hard and slippery surface, which is desirable on country highways.

The speaker's experience has convinced him that a large plant, costing from $5 000 to $8 000, is not necessary in order to construct this pavement successfully, and that equally good results can be obtained from the use of small mixers costing from $1 500 to $2 000.

This pavement without the oil treatment cost about 85 cents per sq. yd., and with an oil seal coat about 90 cents per sq. yd. for a pavement 2½ in. in depth. This cost compares favorably with similar figures for mixing-method pavements in other parts of New York State. The speaker quotes from a statement attributed to a superintendent of highways, of New York State, relative to costs of similar pavements of graded stone material, as follows:

1½ in., California asphalt	$1.20	per sq. yd.
2 in., Topeka	1.20	"
2 in., Warrenite	1.30	"
2 in., Bitulithic	1.60	"

The gravel used in this pavement cost approximately $2.35 per cu. yd., f. o. b. destination, but where material can be obtained near the site of the works, the cost of 85 cents per sq. yd. can be materially reduced.

From observations of mixing-method pavements laid in New York City and vicinity, with graded stone as a mineral aggregate, it is ventured that the percentage of disintegration is as great as, if not greater than, in the bituminous gravel pavement laid during the same period of time. In fact, after 1 year's wear, out of a total of 112 000 sq. yd., or 0.04 of 1%, less than 45 sq. yd. of this gravel mixing-method pavement had disintegrated.

The road which received the treatment of hot oil in a squeegee course was dug up in many places during the year following its completion for investigating purposes, and, where removed, the material thrown back retained its vitality to such an extent that the top course healed itself so that a casual observer could not discover where the pavement had been disturbed.

In designing a country highway, not only the original outlay, but the cost and ease of maintenance should be given proper consideration. Those who have had experience in the upkeep of gravel roads realize

the small expense necessary to retain them in their original condition. Where slight depressions occur from time to time, all that is necessary is to place a sufficient quantity of gravel in these depressions and allow the traffic to consolidate it. In cases where the road has become pitted to such an extent that it requires more extensive treatment, it can be scarified, harrowed, re-rolled, and puddled, and, with the addition of a small quantity of material, can be made as good as the original at a comparatively slight cost, due to the presence of the clay-binding material. This treatment cannot be successfully consummated in broken-stone roads, for as soon as the bond of an old stone macadam road is broken, the material is usually worthless. These peculiarities of the two materials result in a very pronounced saving in the maintenance of gravel roads, as compared with that of broken-stone roads, whether or not they had been subjected to hot oil treatment. The ease of maintenance of the gravel, mixing-method pavement is sufficiently illustrated by the self-healing incident referred to. Mr. Stewart.

The speaker is led to believe that gravel, sand, and clay have not been given the consideration they deserve by the highway engineers of the United States. In a section of the country which possesses clay and sand-bearing gravel, engineers can well afford to investigate its uses for road-making purposes, either as a foundation course, a water-bound top course with or without hot oil treatment, or a top course using the mixing method.

To conclude, the speaker cannot express himself more clearly, relative to the use of gravel, than by referring to the following statement:*

"A well built and well maintained gravel road is preferred by automobilists to macadam roads or even roads paved with more expensive materials. The gravel road is springy and resilient and gives good grip to the tires. It does not produce the jar that exists in riding over a paved road with an unyielding foundation."

This is also true for the man who uses his horse for either business or pleasure.

James Owen, M. Am. Soc. C. E.—This question of sand-clay roads was initiated about 10 years ago at the suggestion of the United States Office of Public Roads, and the writer believes that roads of that type were afterward constructed in South Carolina. The idea was suggested to the writer by his experience in the construction of a speedway, for the surface of which no natural materials were available, so that they had to be manufactured in the best possible manner, and the mixture that had been made in South Carolina was tried. There was available some very good, greasy, gray clay, which was essential for the work, and about 2 miles away there was a very good bed of sand to mix with it. The writer, however, made five or six mixtures Mr. Owen.

* From *Engineering News*.

Mr. Owen.

of the sand and clay before he obtained what might be considered a perfectly consolidated surface. These mixtures were put down in places where there was considerable travel, and thus the best method of surfacing was ascertained.

The speedway was built with the mixture giving the best results; the clay was spread over it in lumps; the sand was then put on top of that, and the two were spaded together and rolled.

One interesting thing happened in these experiments. The mixture was made according to a preconceived idea of the standard surface, and when it was completed it was impossible to get a smooth trotting way for horses. The surface was rolled, but it would not come down, under the conditions prevailing at that time. By accident, a bed of what is known as New Jersey loam was found, and a coating of this, from ¼ to ½ in. thick, was put on, rolled down, and, with the consolidation of the gravel and the clay on top, and the addition of the loam, a perfectly smooth and constant roadway surface was obtained.

It is well to note here that, in the construction of sand-clay roads, strange results may be obtained. In New Jersey there are two classifications of roads; in the north they are of stone, in the south, gravel. In both sections there is an enormous amount of travel, automobile and horse, and the experience of to-day tends to confirm the belief that the gravel roads are much more constant in uniform maintenance, much cheaper to repair, and much more satisfactory to travel on than the stone roads, and here comes in the whole question, as enunciated in this paper, that the original idea of macadam roads as the best means of travel under the old regime, of horse travel and hard steel tire travel, has passed away; that in all localities of the United States there is material at hand which should be used for the purpose for which it is required, and manipulated so that the results are good.

Roadmakers in the future must not rely on stone roads to maintain the desired conditions for the wear and tear of travel.

In the case of the Georgia roads, the climate must be considered, and this may be more serious than is thought at first. There the frost does not penetrate to any great depth; there is rarely any frost, except in the high lands; but that does not affect the maintenance of roads, and, as far as that is concerned, the practice that is applicable to Georgia may not be applicable in the Northern States, where frost often penetrates the ground from 2 to 3 ft.

Experience to-day in New Jersey, however, shows that there is no doubt that it is much more economical to use the natural material, properly manipulated, than to import ideal material to make the ideal road, which in the end does not give ideal results.

JOHN C. KOCH, ASSOC. M. AM. SOC. C. E. (by letter).—Mr. James seems to be under the impression that the writer's investigations and studies of sand-clays were confined to those of a restricted area in North Georgia, and that the sands prevalent were "the Orangeburg sands of the Lafayette formation". For the purpose of making clear that typical sand-clays, representative of the entire State of Georgia, were examined, it may be sufficient to state that analyses were made from numerous samples from 23 counties lying in the Coastal Plain, from 5 counties along the Fall Line, and from 20 counties in the Piedmont Region. The writer has not been able to find any geological account of the existence of the Orangeburg sands of the Lafayette formation in the northern third of Georgia, which is almost altogether of igneous and metamorphic character and is often referred to as the Crystalline Area. According to Professor S. W. McCallie, State Geologist of Georgia, the Lafayette formation is found only on the Coastal Plain, and has probably been confused with certain red sands and gravels of the Eocene and Cretaceous formations, also occurring there. As is generally known, the sand-clays of the Crystalline Area give far better results in road work than those of other formations, the sands of the Lafayette formation being only of minor importance. Mr. Koch.

The road problem of the Coastal Plain along the Atlantic seaboard is how to overcome the difficulties incident to a surface soil composed largely of very fine sand. It is often possible to find a fair supply of clay a few feet below the surface. This may contain from 20 to 60% of fine sand. If such material is used, it generally produces fair results, and is certainly much better than the sand road. With reference to the case where the sands are floury and the clays run high in fine-grained sand, an alleviation of sand-road conditions may easily be made with this material. The writer, however, would hesitate to present this as a desirable or satisfactory criterion for sand-clay roads in general.

The simple field tests described by Mr. James, in mixing sand and clay, are applicable, of course, only to cases where artificial mixtures are to be made. These and similar tests have been considered by the writer, but his experience has been that small spheres of the same mixture gave widely variable results, due to different percentages of moisture held in the material. By heating the spheres on a sand bath at 100° cent. until they are of constant weight, the results will be more uniform.

Mr. Blanchard's suggestion that more detailed information relative to the traffic on the various types of roads discussed in the paper would be of value to road engineers, is concurred in by the writer. Probably no subject in road literature is more difficult of adequate statistical treatment than that of traffic. A traffic census will show great fluctuations in weekly as well as monthly averages, and a census

Mr. Koch.

taken before and after road improvement will usually show a greatly increased traffic after such improvement. The improvement of a road often increases the traffic on that road without, apparently, affecting other roads adjoining it.

TABLE 5.—TRAFFIC ANALYSIS OF SAND-CLAY ROADS.

(Approximate.)

Type of vehicle.	Average Daily Number of Vehicles (Both Ways).											
	Jan.	Feb.	Mar.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
Horse-drawn, total	144	216	261	179	144	144	144	170	288	288	179	104
Buggies	20	30	35	25	20	20	20	25	40	40	25	20
One-horse wagon	60	90	100	75	60	60	60	70	120	120	75	40
Two-horse wagon	60	90	120	75	60	60	60	70	120	120	75	40
Four-horse wagon or heavy machinery	4	6	6	4	4	4	4	5	8	8	4	4
Motor-driven, total	20	20	30	40	40	55	55	40	57	57	38	20
Two-passenger automobiles	10	10	15	20	20	30	30	20	30	30	20	10
Four-passenger automobiles	6	6	8	10	10	15	15	10	15	15	10	6
Six-passenger automobiles	2	2	4	6	6	8	8	8	8	8	6	2
Trucks	2	2	3	4	4	2	2	2	4	4	2	2
Ratio, motor to horse-drawn	0.14	0.09	0.11	0.22	0.28	0.38	0.38	0.23	0.20	0.20	0.21	0.19

Table 5 is presented in order to give an idea of the average traffic conditions prevailing on the sand-clay roads of Georgia. It shows the daily average number of vehicles going to and from a typical county seat or shipping point, over a single road, during each month. Traffic to and from larger cities, of more than 20 000 population, is not considered in this table. On some of the sand-clay roads, the traffic will average less than that given in Table 5, and on about 20% of the mileage the number of vehicles of each class may be three times as many. These figures are based on traffic studies made in many counties of Georgia, and are substantially correct in so far as it is possible to prepare average figures from such data. The ratio of motor-driven to horse-drawn vehicles will undoubtedly steadily increase.

The following memoranda concerning the actual loads carried may be of interest in connection with Table 5:

One-horse vehicle,	average	net	load,	750 lb.	
Two-horse	"	"	"	"	1 500 "
Four-horse	"	"	"	"	3 000 "
Motor trucks,		"	"	"	1 500 "

Climatic conditions in Georgia vary considerably, from the southern portion, not far above sea-level, with a very mild climate, to the northern boundary, in the Blue Ridge Mountains, at an elevation of from 600 to 1 500 ft. above sea-level, with a much more rugged climate.

Generally, the inferior natural sand-clays are found in the southern part of the State, where the climate is very mild. As a rule, the best and most durable sand-clays are found chiefly in the northern half of the State. In the latter section the winters are not severe, but the ground freezes and thaws all through the winter, so that the tendency to disintegrate the sand-clay surface is possibly as severe in the aggregate as the spring thaw in the more severe climate of the Northern States. Mr. Koch.

The writer has examined sand-clay road surfaces during the winter, and has studied the effects of the alternate freezing and thawing, for the purpose of arriving at some basis for determining their suitability for withstanding severe freezing and thawing. On the better sand-clay roads the effect of weeks of freezing and thawing was to produce a thin layer of slushy material, from ½ to 1 in. thick, below which the mixture was quite firm and impervious. In a number of such instances, the writer has cleared off the slushy material and carefully cut down through the surfacing which was almost always found to be very dense and hard and to contain scarcely any more moisture than in summer. Where mixtures of the type classified as "Hard" by the writer (in Fig. 1) were used, on removing the surfacing as described, the roadbed beneath the sand-clay was found to be as dry as it would have been in midsummer, if protected by the sand-clay surfacing. This seems to prove that impermeable mixtures should be used to protect the sub-grade properly, in severe climates. In making examinations of these road surfaces, areas several feet square were taken along the center line as well as in the wheel tracks, so that, as far as possible, freak conditions were eliminated.

Another point which Mr. Blanchard raises is well worthy of further study, namely, the effect of different percentages of sand-clay mixtures retained on the 10-mesh sieve, when such mixtures are used under various climatic and traffic conditions. The writer has not had opportunity to study a sufficient number of such cases to feel warranted in stating any conclusions at this time. It is safe to say, however, that, in a sand-clay mixture, the greater the proportion of material coarser than that retained on a 10-mesh sieve, the greater its durability and the more suitable its use for service in a very cold climate, provided, of course, that the coarse material is firm and hard and similar to quartz or other hard rock.

The writer agrees with Mr. Blanchard that many highway engineers will disagree with him in dividing gravel and sand on the basis of the material retained on or passing the 10-mesh sieve. The writer has used the 10-mesh sieve arbitrarily as the dividing line between sand and gravel. The following is the definition* of gravel as

* American Civil Engineers' Pocket Book, p. 169.

Mr. Koch. officially adopted by the American Railway Engineering and Maintenance of Way Association:

"Gravel is defined as small worn fragments of rock occurring in natural deposits, that will pass thru a 2½-in. ring and be retained upon a No. 10 screen."

In the writer's experience in testing sand-clays, the 10-mesh sieve has been very satisfactory as a basis for separating the coarser material from the finer. If the 4-mesh sieve had been used in analyzing the coarser materials, probably not more than 1% of the samples examined would have left any gravel on this sieve. Practically all the material classified by the writer as gravel was larger than 10-mesh size and smaller than 4-mesh size, with the single exception of Sample No. 223. This sample contained 34% of material coarser than 10-mesh size, and one-half of this was larger than 4-mesh size.

Mr. Stewart mentions the application of hot oil to the roads of the Parkway Systems of Greater New York, which were built of Hudson cementitious gravel. Similar treatment has been proposed for sand-clay roads in certain counties of Georgia, but it has not been attempted on account of the relatively great expense. The cost of a seal coat of hot oil is as great as the total cost of the sand-clay surfacing, and the only advantage that could be claimed to accrue from the additional expenditure would be a slight diminution of the dust from automobile travel. The quantity of dust raised on sand-clay roads by automobiles depends largely on the character of the construction. Well-constructed roads of a good class of this material yield from one-fifth to one-tenth of the dust that is raised on a water-bound macadam road under the same traffic conditions.

Mr. Owen's experience with artificial mixtures of sand and clay illustrates a common difficulty in making such mixtures. If natural sand-clay mixtures are available locally, but are not suitable for use alone, because of a deficiency in either the sand or clay content, this defect may be corrected by the addition of the proper quantity of sand or clay. This method of treatment will generally be found to be more economical than an attempt to build up an artificial mixture by using sand as a basis and adding the proper quantity of clay and mixing. Occasionally, it may be possible to combine two natural mixtures in which an excess of sand occurs in one and an excess of clay in the other. To determine the proportions in which such materials should be used, one may make analyses of the materials and plot curves showing their composition. Usually an inspection of these curves will suggest approximately the proportions in which the materials should be used to obtain the best results. Trial mixtures of the two materials may be studied by plotting the analyses of such mixtures, using as a criterion by which to judge their value, the analyses of sand-

clays of known value. Fig. 1 has been used in this manner by the writer with excellent results. Mr. Koch.

The advantage of using a natural sand-clay as the basis for artificial mixtures lies in the fact that the two materials are somewhat thoroughly mixed, and the incorporation of the additional sand or clay required will be much more easily effected than where none of the mixing has been previously performed by Nature. In making an artificial mixture, using sand as a basis and adding the requisite quantity of clay, the latter forms in clods and balls, and thorough incorporation of the two is accomplished only by a long-continued process of puddling.

The effect of climatic conditions on sand-clay roads has been suggested by Mr. Blanchard and Mr. Owen. In regard to the effect of the severe weather conditions of the Northern States, the writer has no definite information. The following mileages of sand-clay roads were reported* in 1909 for the States named:

Connecticut	1 214
Delaware	6
Georgia	4 326
Idaho	398
Iowa	575
Kansas	202
Michigan	2 381
Minnesota	1 051
Oregon	345
Rhode Island	6
South Dakota	129
Washington	1 223
Wisconsin	1 013
Total, leaving out Georgia.	8 543

From the foregoing, it appears to the writer that there is a considerable mileage of roads of this type in use in the Northern States, and that since 1909 there should have been developed among road builders in the States named a large amount of valuable information regarding the use and possibilities of sand-clay road construction. The total mileage of such roads in the United States reported for 1909 was 24 601, so that more than one-third of the entire mileage reported in that year was in what may be called the Northern States.

The writer wishes to call attention to the experiences of several counties, that are of interest because of the great contrast they afford in the economy of sand-clay road construction as compared with water-

* *Bulletin* No. 41, U. S. Office of Public Roads, p. 42.

Mr. Koch. bound macadam. Therefore, the following examples are presented for consideration.

Habersham County, Ga.—A macadam road was laid in 1909, if the writer has been correctly informed, at an expense of about \$2 000 for a length of 2 000 ft. This road was 25 ft. wide, and had field-stone curbs. The macadam was about 5 in. thick after rolling. In 2 years the road had become so rough as to be avoided by the traffic whenever the parallel side streets were not too muddy. In 1912 a new road was built of a natural sand-clay mixture, extending from the end of the old macadam section mentioned, to the railroad station at Clarkesville. This was curbed with field stone, and cost \$1 200 per mile. It is smoother after a year's service than the macadam road ever was, and is easily kept in excellent repair on account of the large quantity of good sand-clay close at hand.

Hall County, Ga.—In the period, 1905 to 1908, this county established quarries, installed machinery of all sorts for road-making, and spent about \$40 000 in building water-bound macadam roads. The rock used was a local gneiss. The roads, for the most part, were surfaced 5 in. thick and for a width of 20 ft. The total length actually constructed was between 8 and 10 miles, as nearly as the writer can learn. The total road mileage of this county is about 900, of which probably 200 miles would serve 50% of the traffic. From this it can be readily seen how impossible it would be to construct the most important roads with macadam. The average annual income from taxes for road improvement in this county is about \$15 000.

In the fall of 1911 almost all the macadam roads which had been constructed were in such bad condition that reconstruction seemed absolutely necessary. To reconstruct these roads with macadam was a sheer financial impossibility, as the county was still in debt for the original construction and equipment. Temporary repairs were made, which rendered the roads passable that winter. In the fall of 1912, the writer was consulted by the county authorities as to what could be done to improve road conditions, and as to the adoption of a definite low-cost programme that was financially feasible. After an examination of the macadam and other roads, all of which were in wretched condition, especially the former, the writer recommended the adoption of the sand-clay road wherever that material could be used without excessive expense. After sampling a number of large areas of natural topsoil sand-clay, the writer found that, with few exceptions, excellent natural mixtures were readily available with short hauls; so that almost all the important roads were favorably situated for reconstruction with these mixtures. It was also found that the deep holes in the macadam roads could be filled with the sand-clay, which bonded well with the macadam, and, at comparatively small expense, many of these roads were thus repaired. The remainder was

plowed up and loosened and then a 10-in. layer of sand-clay was placed over it and compacted by the traffic. Since then the county has carried out a consistent policy of repairing old macadam with sand-clay until the macadam is worn out, and making all new construction of sand-clay. Under this new policy, 20 miles of new sand-clay roads have already been constructed, at an average cost of about $425 per mile; the old macadam is still giving some service, and the improved roads are passable at all seasons of the year. All the expenses incident to this new policy have been met by current revenues. Mr. Koch.

Clarke County, Ga.—The experience of this county was somewhat similar to that of Hall County, except that sand-clay construction has been in use here for nearly 7 years. During the period, 1905-08, Clarke County spent about $50 000 on the construction of 7 or 8 miles of water-bound macadam roads. This had been accomplished in a little more than 3 years. Then it became evident that the cost of maintenance was a far more serious item than had been thought. It was so great that it soon became clear that macadam was too expensive an investment for the county's revenues.

In 1907, Mr. W. H. Holman, Superintendent of County Roads, began experimenting with natural mixtures of sand-clay, taken from the surface of cultivated fields, as a road-surfacing material. This material is often referred to as "topsoil". The experiments were so successful that topsoil was soon adopted as the surfacing material for all roads, and by 1913, practically all the important roads of the county had been surfaced with it, for a total length of 100 miles. This large mileage was built with current revenues (approximately $20 000 annually), without a bond issue or the assumption of a heavy burden of taxation. This splendid work has been brought about largely as a result of the untiring efforts of C. M. Strahan, M. Am. Soc. C. E., who, as County Engineer, has been identified with this road work for many years.

The average annual cost per mile of the macadam roads built by this county, including bond interest, sinking fund, and maintenance, was found to be about $1 200 (neglecting the fact that after from 4 to 6 years of service they were covered with topsoil to prevent complete raveling and destruction).

The cost of sand-clay roads in this county has averaged about $500 per mile, so that the annual cost of 1 mile of macadam (neglecting depreciation entirely), was sufficient to build 2.4 miles of new sand-clay road. The cost of maintenance of sand-clay roads of good quality is very low, averaging $5 per mile per annum in this county. Poorer qualities of sand-clay can be kept in excellent condition at an expense of from $10 to $30 per mile per annum depending on its quality and the character of maintenance required.

Mr. Koch. In conclusion, it should be stated that the specific cases just described are presented as concrete examples of typical experiences in many parts of the South, and that these cases are not referred to in any spirit of criticism, but for the purpose of emphasizing the fact that, in many parts of the United States in which the principal, but not the only, difficulty has been poverty, sand-clay offers the only feasible solution of the road problem. There are many counties where the mileage of public roads is so great that there is a mile of road for every three to ten voters. It is especially for such sparsely settled districts that the improved low-cost road is a necessity. Though the sand-clay road is adapted to the heavy traffic of much more densely settled communities than those indicated, it also offers to sparsely settled communities hope of improved roads without an excessive burden of taxation.

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Paper No. 1307

SHEARING STRENGTH OF CONSTRUCTION JOINTS IN STEMS OF REINFORCED CONCRETE T-BEAMS, AS SHOWN BY TESTS.*

By Lewis J. Johnson, M. Am. Soc. C. E., and John R. Nichols, Jun. Am. Soc. C. E.†

With Discussion by Messrs. L. J. Mensch, J. P. Snow, D. Gutman, Elwyn E. Seelye, Alfred B. Heiser, Henry G. Raff, G. E. Doyen, Thomas H. Wiggin, Alexis Saurbrey, and Lewis J. Johnson and John R. Nichols.

In reinforced concrete construction, the importance of lessening the cost of forms has long been recognized. A recent effort in this direction has resulted in a system of construction in which columns and stems of beams and girders are cast on the ground. These units, when hard, are hoisted into place and set in mortar or grouted. The concrete skeleton or framework thus erected supports the forms for the floor-slab, which is then cast in place. The slab rests on the beam and girder stems, and is secured to them by the customary web reinforcement—stirrups and bent-up or trussed tension rods.

In the ordinary beam and girder type of floor, the slab, besides receiving the live load and distributing it to the beams, performs the important function of acting as compression flange for the beams and girders. In the system just described, however, a horizontal construction joint exists between the slab or flange and the stem below, due to the lapse of several days between the casting of the two. This forces to the front the question whether this joint can transmit safely the horizontal shear necessary to make the slab and stem act together as a beam, as in an ordinary monolithic floor.

* Presented at the meeting of April 2d, 1913.

† Now Assoc. M. Am. Soc. C. E.

This paper is a record of tests which, among other things, seems to show clearly that, under conditions readily met in practice, the answer to this question is in the affirmative. These results are so strikingly at variance with ideas previously widely held that a full account of the conditions of the tests must be included.

These tests were undertaken under the following circumstances:

In December, 1911, a Boston company had contracts for two buildings on which it was proposed to use the unit construction. J. R. Worcester and L. J. Johnson, Members, Am. Soc. C. E., were retained as Consulting Engineers on the general design of the unit construction work. At their instance, a series of tests was undertaken by the writers, in the laboratory of the School of Engineering at Harvard University, to investigate the strength in shear of the horizontal joint between stem and floor-slab necessarily involved in this type of construction.

As work on one of the two buildings had to go on without delay, it was agreed, pending the completion of the tests, to introduce the precautionary measure of increasing the shear resistance of the construction joints by corrugating the tops of the stems in a manner similar to that shown in Fig. 3 (Types *A* and *B*) and in Fig. 4. Thus work was begun. The tests soon made it clear, however, that the corrugations were unnecessary, and they were omitted in subsequent construction.

Objects of Tests.—The specific objects of the tests were three:

1.—To determine whether the type of construction going into the building, involving the corrugated joint between the stem and the slab, was safe;

2.—To determine whether it would be safe to dispense with the corrugations and use instead a smooth, that is, an ordinary, untreated joint; and

3.—To discover, if possible, how high a shearing stress such a smooth joint can resist.

Types of Test Beams.—For the tests, three types of beams, *A, B,* and *C,* were made.

Type *A* beams were designed by the Boston company primarily for the first and second objects stated. Accordingly, their design was similar to that of the beams going into the buildings.

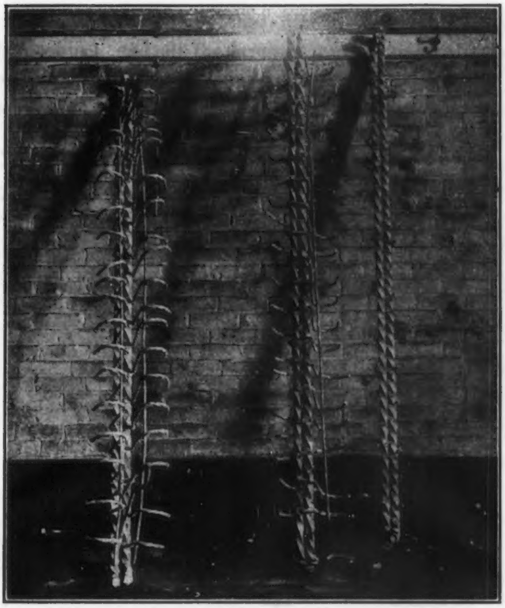


FIG. 1.—REINFORCEMENT OF THREE TYPE *B* BEAMS AFTER THE TEST. THE CUTTING OR REMOVAL OF THE STIRRUPS OCCURRED IN DISENGAGING THE STEEL FROM THE CONCRETE.

FIG. 2.—TOP SURFACE OF STEM OF BEAM NO. 16, SHOWING QUALITY OF SURFACE OF A "SMOOTH" JOINT.

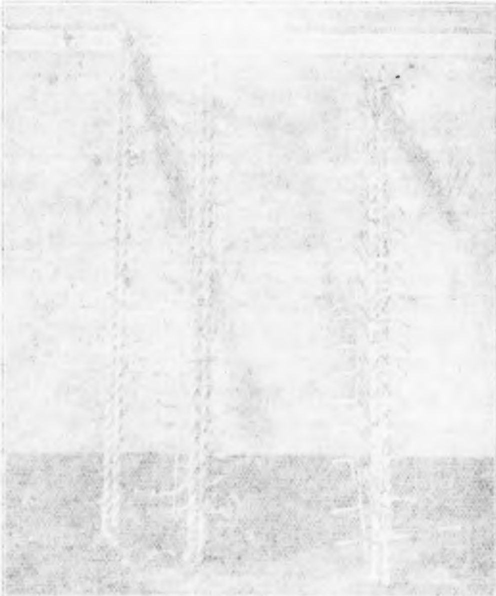


FIG. 1.—[illegible] TYPE 2 BEAMS AFTER TEST [illegible]

FIG. 2.—TOP SURFACE OF SLAB OF BEAM No. 10, [illegible]

Type *B* beams, designed by the authors, were intended primarily for the third object, and all the stresses were kept low in comparison with the shearing stress in the stem, in order that the shear on the joint might reach a high value before the beam failed from any other cause.

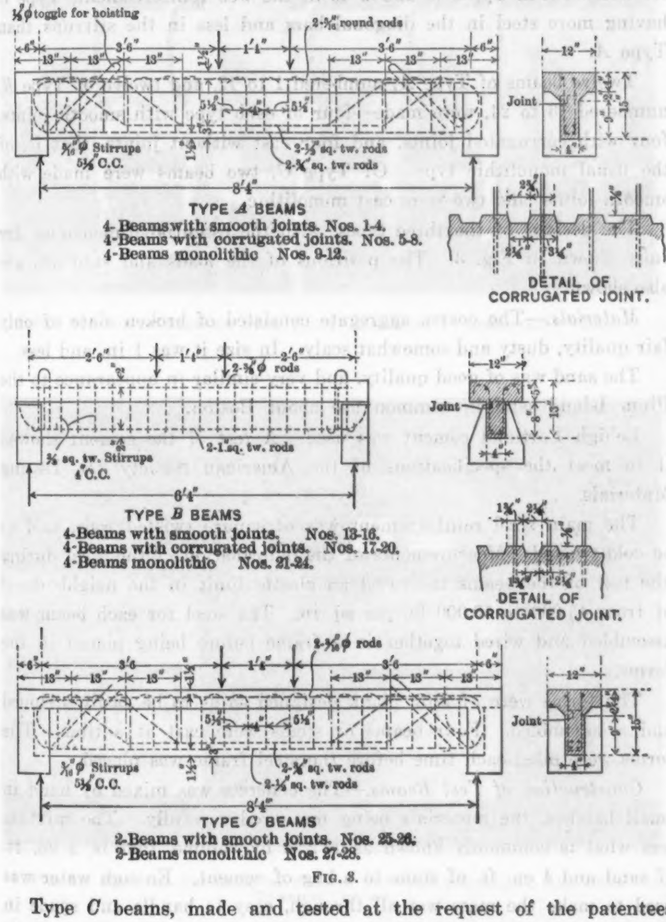


FIG. 3.

Type *C* beams, made and tested at the request of the patentees of this system of unit construction, were four in number (Nos. 25 to 28). They differed from the Type *A* beams in putting the larger

rods of the main reinforcement in the upper of the two layers and turning them up for diagonal web reinforcement, instead of having the smaller rods in this position. They differed further in having smaller rods for the stirrups. In other words, the significant difference between Types *A* and *C* is in the web reinforcement, Type *C* having more steel in the diagonal bars and less in the stirrups than Type *A*.

Twelve beams of Type *A*, numbered 1 to 12, and twelve of Type *B*, numbered 13 to 24, were made—four of each type with smooth joints, four with corrugated joints, and four cast without joints, that is, of the usual monolithic type. Of Type *C*, two beams were made with smooth joints, and two were cast monolithic.

The designs of the three types, with the essential dimensions, are fully shown in Fig. 3. The positions of the loads and supports are also shown.

Materials.—The coarse aggregate consisted of broken slate of only fair quality, dusty and somewhat scaly. In size it was 1 in. and less.

The sand was of good quality, and very similar in appearance to the Plum Island sand in common use about Boston.

Lehigh Portland cement was used. A test of the cement showed it to meet the specifications of the American Society for Testing Materials.

The main steel reinforcement was of square twisted rods, said to be cold-twisted. Measurements of the deformation of the steel during the test of the beams indicated an elastic limit in the neighborhood of from 45 000 to 50 000 lb. per sq. in. The steel for each beam was assembled and wired together in a frame before being placed in the forms.

The forms were of 2-in. plank designed so as to be easily stripped and re-assembled. Four beams or stems were cast at a time. The forms were oiled each time before the steel frame was placed.

Construction of Test Beams.—The concrete was mixed by hand in small batches, the materials being measured carefully. The mixture was what is commonly known as 1 : 2 : 4 by volume, that is, 2 cu. ft. of sand and 4 cu. ft. of stone to a bag of cement. Enough water was used to make the mass wet all through, easy to handle and spade in the forms, but not sloppy.

In order to get the main reinforcing rods into the narrow 4-in.

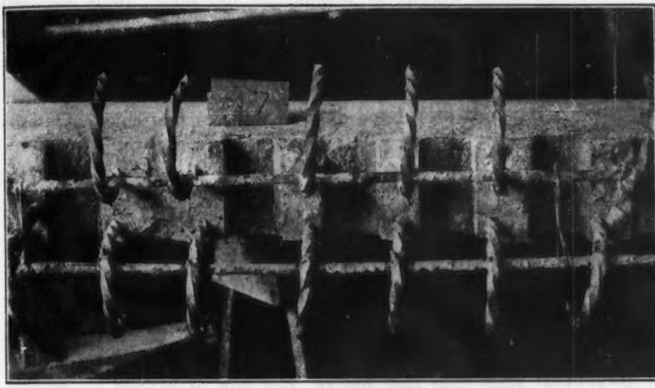


FIG. 4.—TOP OF STEM OF BEAM NO. 17, SHOWING A "CORRUGATED" JOINT.

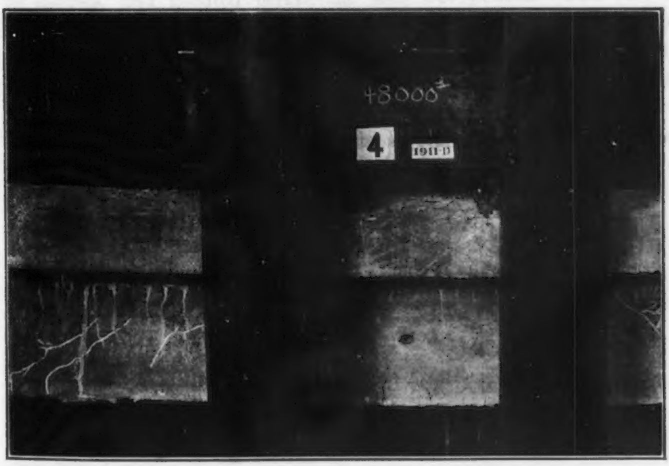
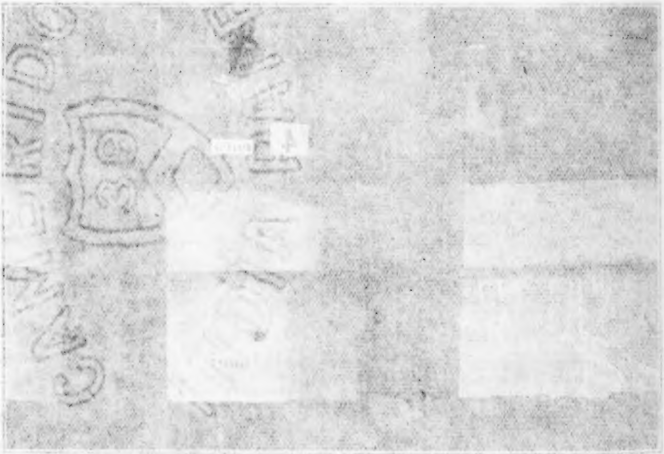


FIG. 5.—SHOWING CRUSHING OF FLANGE BETWEEN THE LOADS, AND OPEN TENSION CRACKS.



stem, they had to be placed so close to one another (Fig. 1) that the stone could not be tamped or spaded down between them. Clear mortar, mixed 1:2, was accordingly used in the bottom of the stem, up to the top of the main rods. The concrete was placed on top of the mortar and puddled with trowels and small rods, in order to eliminate voids and secure thorough bond with the steel.

The top of the stem was leveled off in the case of the "smooth joint" beams, but not troweled. The quality of the surface is shown clearly in Fig. 2.

To make the corrugations shown in the design and in Fig. 4, beveled blocks of wood were pressed down into the forms, flush with the top of the stem, and the concrete between the blocks was leveled off.

The surfaces of the joints were not subsequently brushed, or scraped, or treated in any way, except that they were thoroughly wet just before placing the concrete of the slab. The stems of the jointed beams of Types *A* and *C* were set in concrete, end-supporting blocks, in pockets, and grouted before the slab was poured. The bottom of the slab, when it was poured, came in direct contact with the tops of these blocks. The beams of Type *B*, however, were set in mortar in the concrete, end-supporting blocks, after the slab was hard. The general appearance of the three types is shown by Figs. 6, 7, and 8.

Table 1 gives the dates on which the several stems, slabs, and beams were cast.

TABLE 1.—DATES OF CASTING TEST BEAMS.

January 8, 1912	Stems 1, 2, 3, 4	(smooth).
" 9, 1912	" 5, 6, 7, 8	(corrugated).
" 10, 1912	" 13, 14, 15, 16	(smooth).
" 11, 1912	" 17, 18, 19, 20	(corrugated).
" 13, 1912	Beams 9, 10, 11, 12.	
" 17, 1912	Slabs 1, 2, 3, 4	Age of stem, 9 days.
" 18, 1912	" 5, 6, 7, 8	" " " 9 "
" 19, 1912	" 13, 14, 15, 16	" " " 9 "
" 21, 1912	" 17, 18, 19, 20	" " " 10 "
" 23, 1912	Beams 21, 22, 23, 24.	
" 25, 1912	Stems 25, 26	(smooth).
" 27, 1912	Beams 27, 28.	
" 29, 1912	Slabs 25, 26	Age of stem, 4 days.

The forms were usually removed in from 20 to 24 hours. The beams were stored in the basement where they were made, and no special provision was made for keeping them wet.

The workmanship was of commercial grade, perhaps somewhat, though not significantly, better than is secured in most reinforced concrete work. The workmen were carpenters experienced in concrete work, one of them a general foreman for reinforced concrete building construction. Although no laboratory refinements were used in the manufacture of the test beams, the workmen knew the purpose for which they were being made, and, accordingly, were careful. On the other hand, they were pushed to a time schedule of four beams a day, which required the utmost speed of which they were capable, and which in the later stages was impossible of attainment, due to the slow hardening of the concrete. To this hurry are due slight imperfections in workmanship which appeared later. These imperfections were of three kinds:

1.—The steel was sometimes slightly misplaced;

2.—In a few of the Type *B* beams it was observed that the slab was not exactly perpendicular to the stem; and

3.—A slight curvature of the beam in plan was noticed in one or two cases.

Testing.—The beams were tested in two groups. The first tests were made on February 22d and 23d. Eleven beams were tested at this time, which sufficed to show that the type of construction going into the building was safe, and that the joints did not need to be corrugated. The remaining beams were reserved to allow the concrete to become stronger. These were tested during the week of April 15th, except Beams 21, 26, and 28 which were tested on May 4th.

The beams were tested in the 200 000-lb., Olsen testing machine in Pierce Hall, Harvard University. The beam supports consisted of rockers, which ensured a vertical reaction at the ends of the beam in all cases except that of Beam 2. During the test of this beam, the supports rocked back to the flat portion of the rocker, giving the reaction sufficient horizontal component toward the center of the span to cause the concrete supporting block to spall off with a vertical crack above the rocker. This may have contributed somewhat to the high result with this beam, but the writers do not believe that the error involved is large, and the beam has accordingly been averaged with the rest without modification.

Between the rocker and the concrete was interposed in each case a steel plate and a cushion of $\frac{3}{8}$-in. whitewood to distribute the pressure.

FIG. 6.—TYPE *A*, BEAM NO. 6.

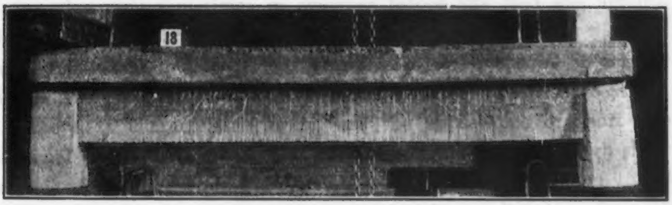


FIG. 7.—TYPE *B*, BEAM NO. 18.

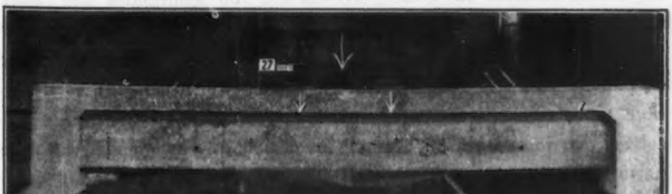


FIG. 8.—TYPE *C*, BEAM NO. 27.

The load was transmitted from the movable head of the testing machine by a pair of 8-in. I's to two steel rollers, each 8 in. from the center of the span, thence down through a steel plate and white-wood cushion to the top of the beam. This arrangement, which is shown on top of beam No. 27, Fig. 8, worked satisfactorily in all cases.

The rate of application of the load was from 2 500 to 3 500 lb. per min. for beams of Types *A* and *C*, and from 5 000 to 5 500 lb. per min. for Type *B* beams, the increase in rate being due to the greater stiffness of the shorter beams. For some beams the load was removed after reaching the maximum, and re-applied at a rate, for the second and subsequent applications, some four times as fast as for the first loading.

Phenomena of Tests.—The behavior of the beams during the application of the load was remarkably uniform. Vertical cracks at the lower edge were sometimes in evidence before any load was applied, but usually appeared near the center of the span when the load was from 5 000 to 10 000 lb. These were often spaced at regular intervals of from 4 to 6 in., corresponding probably to stirrup locations.

The next characteristic event was the appearance of short fine diagonal cracks at mid-height of the stem in the region of high shear outside the loads. They were first observed at a load of from 15 000 to 20 000 lb. As no extraordinary pains had been taken to make such cracks readily visible, the actual break in the concrete may have come earlier.

As the load increased, these short cracks extended both up and down; and sometimes, but not always, joined the vertical cracks previously observed. They became at the same time more numerous and wider.

Up to this point, the behavior of the three types of beams was substantially the same. As the load approached the maximum, however, the phenomena varied. In the Type *A* beams, one or two vertical cracks developed at the center, widening and extending upward toward the fillets. Coincident with the load reaching the maximum, the top surface of the beam began to flake up between the loads, and short horizontal cracks appeared on the side of the slab, indicating crushing of the concrete. This is shown in the photograph of the center of Beam 4, Fig. 5.

Beams 9 and 12 failed prematurely by spalling off the lower corner of the stem within the concrete supporting blocks, just outside of the curved ends of the lower main rods, as shown by Fig. 9.

In the Type *B* beams, the development of the diagonal cracks was the most important feature, as the load approached the maximum, until it was noticed that the main rods were slipping. In some cases the ends of the rods were uncovered and a crack between the end of the rod and the concrete was direct evidence of the slipping. Slipping was also indicated by the cracking and flaking of the concrete at the end of the beam caused by the wedging action of the twisted rods. Both kinds of evidence are presented in Fig. 10.

After the development of numerous diagonal cracks, after the rods began to slip, and after the maximum load was passed, the surface of the stem frequently began to flake off, indicating crushing in a diagonal direction. Fig. 11 shows this phenomenon clearly.

After the maximum load had been passed with a few of the beams, the load was taken off and re-applied several times. The toughness of reinforced concrete construction is well illustrated by the high loads these beams were able to carry even when they had been badly shattered by repeated loadings.

Beams 15, 16, 17, and 23 spalled off outside the curve of the main rods in the supporting blocks, but it is probable that slipping of the rods occurred before the spalling, and was the direct cause of it.

In some of the Type *B* beams, a fine horizontal hair crack was observed at the end of the beam, level with the bottom of the slab, when the loading was well advanced. This, however, appeared in the monolithic as well as the jointed beams; it seemed to have no connection whatever with the failure of the beam, and no significance is attached to it. It was doubtless one of the local effects of the heavy concentration of the loads.

In the Type *C* beams, as the load approached the maximum, the diagonal cracks opened rapidly, and the surface of the stem flaked off, followed closely by crushing of the concrete at the top of the flange at mid-span. The flaking of the stem-surface was particularly marked where the truss rods turned up, and is shown in Fig. 12.

The numerical results of the tests are set forth briefly in Table 2.

Discussion of Test Data.—Columns 1, 2, and 3 of Table 2 need no explanation. Columns 4 and 5 give the ages of the stem and slab,

FIG. 9.—PREMATURE FAILURE OF BEAM NO. 9 AT THE END. NOTE DEVELOPMENT OF TYPICAL DIAGONAL CRACKS. THE CHALK FIGURES ON THE BEAM INDICATE THE TOTAL LOADS, IN THOUSANDS OF POUNDS, WHEN THE CRACKS WERE FIRST OBSERVED.

FIG. 10.—EVIDENCE THAT THE RODS SLIPPED IN AT LEAST SOME OF THE TYPE *B* BEAMS. NOTE THE OPENING ABOVE THE ENDS OF THE HOOKS OF THE REINFORCING RODS.

TABLE 2.—Numerical Results of Tests.

Type.	Joint.	Beam No.	Stem age, in days.	Slab age, in days.	Maximum load.	Computed Unit Stresses at Maximum Load. Based on design. Steel.	Based on design. Concrete.	Based on design. Shear.	On measurements.* Steel.	On measurements.* Concrete.	On measurements.* Shear.	Apparent immediate cause of failure.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
A	Smooth.	1	46	37	49 500	57 400	2 770	555	59 700	2 950	578	Comp. in flange.
		2	46	37	54 300	63 000	3 040	610	66 000	3 270	638	" " "
		3	101	92	50 960	59 000	2 850	572	58 600	2 820	568	" " "
		4	45	36	48 000	55 600	2 690	539	55 500	2 680	538	" " "
		Average......			50 700	58 750	2 840	569	59 950	2 930	580	
	Corrugated.	5	44	35	41 000	47 500	2 300	460	48 700	2 400	472	Comp. in flange.
		6	101	92	48 300	56 000	2 700	542	58 400	2 890	565	" " "
		7	100	91	52 100	60 500	2 920	585	61 500	2 990	594	" " "
		8	101	92	48 700	56 500	2 730	547	59 200	2 940	573	" " "
		Average......			47 500	55 100	2 660	534	56 950	2 805	551	
	Monolithic.	9	41	41	50 000	58 000	2 800	561	59 400	2 920	575	Broke in support.
		10	41	41	52 400	60 800	2 930	589	61 100	2 950	592	Comp. in flange.
		11	97	97	51 000	59 200	2 860	573	59 200	2 860	573	" " "
		12	41	41	45 800	53 100	2 560	514	53 900	2 620	522	Broke in support.
		Average......			49 800	57 800	2 790	559	58 400	2 840	566	
B	Smooth.	13	43	34	50 000	40 600	2 440	676	39 700	2 350	661	Main rods slipped.
		14	97	88	50 000	40 600	2 440	676	43 200	2 720	719	" " "
		15	98	89	55 700	45 200	2 720	754	46 100	2 820	771	" " "
		16	97	88	51 400	41 700	2 510	695	42 200	2 560	702	" " "
		Average......			51 800	42 000	2 530	700	42 800	2 610	713	
	Corrugated.	17	97	87	59 900	48 600	2 930	810	47 500	2 820	792	Main rods slipped.
		18	97	87	63 250	51 400	3 090	856	50 800	3 030	846	" " "
		19	97	87	63 600	51 600	3 110	860	53 200	3 230	888	" " "
		20	42	32	55 600	45 100	2 720	752	46 500	2 870	776	" " "
		Average......			60 600	49 200	2 960	820	49 500	3 000	826	
	Monolithic.	21	102	102	57 140	46 400	2 790	773	46 900	2 840	781	Main rods slipped.
		22	85	85	55 400	45 000	2 710	750	44 000	2 610	734	" " "
		23	87	87	55 650	45 200	2 720	752	44 700	2 670	744	" " "
		24	84	84	54 000	43 800	2 640	730	43 300	2 590	722	" " "
		Average......			55 550	45 100	2 710	751	44 700	2 680	745	

* Taking into account the actual placing of the steel.

TABLE 2.—(*Continued.*)

Type.	Joint.	Beam No.	Stem age, in days.	Slab age, in days.	Maximum load.	Computed Unit Stresses at Maximum Load. Based on design. Steel.	Concrete.	Shear.	On measurements.* Steel.	Concrete.	Shear.	Apparent immediate cause of failure.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
C	Smooth.	25	29	25	42 500	50 800	2 590	492	51 100	2 620	495	Comp. in flange.
		26	100	96	43 750	52 300	2 670	506	53 700	2 780	520	Comp. in stem.
		Average......			43 100	51 650	2 630	499	52 400	2 700	508	
	Monolithic.	27	27	27	45 700	54 600	2 790	529	55 000	2 830	533	Comp. in stem.
		28	98	98	47 000	56 200	2 870	544	57 700	2 990	559	" " "
		Average......			46 350	51 400	2 830	536	56 350	2 910	546	

* Taking into account the actual placing of the steel.

respectively, when the beam was tested. The difference between the two in any one line, gives the age of the stem when the slab was cast. Column 6 gives the total load carried by the beam, exclusive of its own weight.

The unit stresses shown in Columns 7 to 12 are computed from the maximum load, as given, on the assumptions ordinarily used in the design of reinforced concrete beams. These include the assumption of straight line or planar distribution of stress in the concrete, and a value of 15 for the ratio of modulus of elasticity of steel to that of concrete. Columns 7, 8, and 9 are based on a beam cross-section in accordance with the design. It was observed, however, that the position of the main rods varied somewhat from that intended. The actual position of these rods was carefully ascertained, and the stresses on the actual cross-section thus determined are given in Columns 10, 11, and 12.

The columns headed "Steel" record the unit tension in the main rods. Those headed "Concrete" give the maximum unit compression in the concrete at the top of the slab. The columns headed "Shear" give the unit horizontal and vertical shear in the stem. This is also the intensity of the shear on the joint between the stem and the slab

FIG. 11.—DIAGONAL CRUSHING OF STEM UNDER LOADING CONTINUED AFTER MAXIMUM LOAD HAS BEEN PASSED.

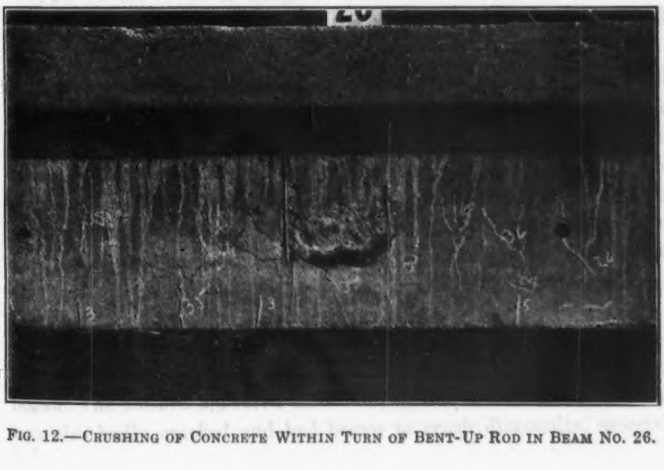


FIG. 12.—CRUSHING OF CONCRETE WITHIN TURN OF BENT-UP ROD IN BEAM NO. 26.

Fig. 11.—Diagonal Cracking of Beam … Without Stirrups After Maximum Load Had Been Reached.

Fig. 12.—Crushing of Concrete Within Turn of Bent-Up Rod in Beam No. 26.

(in those beams which have a joint), the strength of which it is the purpose of these tests to investigate, and the intensity of the diagonal tension in the web, as ordinarily computed.

From the unit shear may be obtained, if it is desired, the unit adhesion on the two straight rods by multiplying the shear for Type *A* beams by $\frac{2}{3}$, for Type *B* beams by $\frac{1}{2}$, and for Type *C* beams by 1. It will be observed that both the adhesion and the shear ran high, the former to 559 lb. per sq. in., the latter to 888 lb. per sq. in.

Cause of Failure.—Type A.—The last column of the table gives what appeared to be the immediate cause of failure. It is often difficult to determine with certainty the one feature in the design of a beam which would most need strengthening in order to increase the strength of the beam. Though most of the failures of Type *A* beams are ascribed to crushing of the concrete, for the reason that the crushing was the immediate visible forerunner of the maximum load attainable in the testing machine, yet it seems certain that in all cases the elastic limit of the steel was overstepped, leading to a rising neutral axis, a diminishing compression area, and a consequent premature crushing. To what extent the stretch of the steel was responsible for failure cannot be told, but it probably was the prime cause in all cases. It is not essential to this investigation to know.

Two of the Type *A* beams failed in the supports, as noted, by spalling off the lower corner of the stem below and outside of the curve of the lower rods.

Cause of Failure.—Type B.—The primary cause of failure of the Type *B* beams was probably slipping of the main tension rods, although this was not observed in all cases before the maximum load was attained. The further destruction of the stem and slab by cracking and crushing under continued application of the load cannot be regarded as the cause of failure. There is every probability that, if the same quantity of steel had been secured with a larger number of smaller rods, the strength of all the Type *B* beams would have been materially increased.

Cause of Failure.—Type C.—In Beam 25 it was not observed whether the maximum load was passed before the crushing of the flange began. The failure appeared to be due to this crushing, but the stem was also badly cracked and had begun to crush diagonally, especially

in the lower turn of the truss rods. Furthermore, the elastic limit of the main steel was doubtless overstepped.

In the case of the other three beams of Type C_1, the maximum load had clearly been reached before the flange began to crush, but the crushing of the stem was coincident with or preceded the maximum load. How far the failure of the concrete in bearing at the turn of the truss rods, and the consequent straightening and yielding of the rods themselves, reduced the effectiveness of the web reinforcement, and thus became the primary cause of failure, cannot be told. Neither can the effect of passing the elastic limit of the main rods be estimated.

Further evidence that the steel was the limiting factor in the strength of the beams lies in the fact that there is only a very slight difference in strength between beams 40 days old and those 90 days old. If the concrete were the primary cause of failure, this difference should have been more marked.

General Results.—Horizontal shear or slipping on the joint was looked for in all the beams with great care, but was never observed. In fact, the jointed beams, both smooth and corrugated, withstood the test as well as the monolithic. There was not the slightest evidence that the joint contributed in any way to the failure of the beams, though the shearing stress on the joint reached values greatly in excess of four times the 120 lb. per sq. in. allowed by the Joint Committee in beams reinforced for shear.

Three interesting load-deflection curves, made by an autographic device on the testing machine, are presented in Fig. 13.

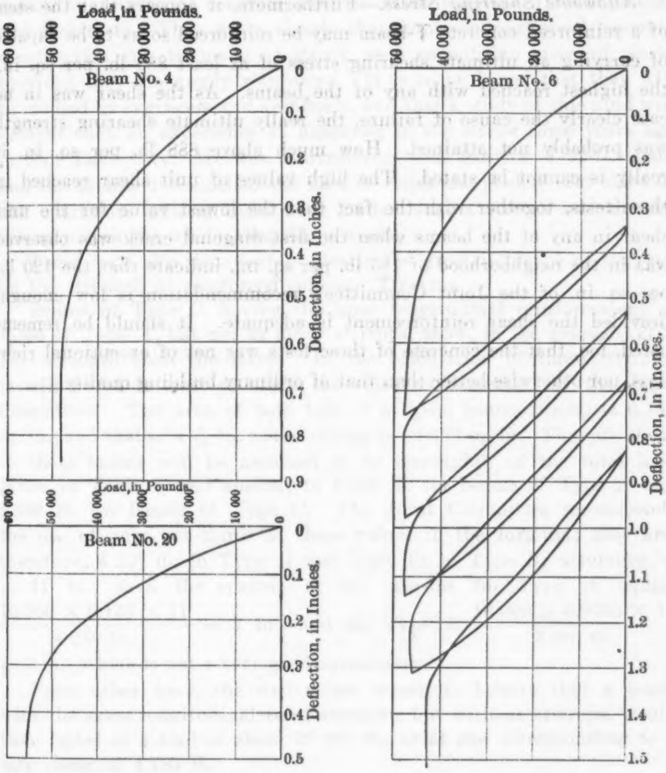
Conclusions.—In view of the striking uniformity, both in the general behavior of the beams during the test and in the numerical results attained, these tests would seem to justify the following conclusions:

1.—That the type of construction used on the buildings was safe;

2.—That the joints did not need to be corrugated; and

3.—That a smooth or unroughened joint, constructed in the same manner as the joints tested in a beam suitably reinforced for shear (or diagonal tension), is capable of transmitting with ample factor of safety any shear that could safely be permitted in a monolithic beam of the same cross-section and having the same reinforcement.

One Important Bearing on Engineering Practice.—It may reasonably be inferred from the results of these tests that in the ordinary monolithic floor construction, where adequate web reinforcement is provided with sufficient anchorage at the top, the slab need not be



Two Typical Load-deflection Curves. Note the Greater Stiffness of the Type *B* Beam, No. 20.

Load-deflection Curve for Beam No. 6. Load Removed and Reapplied Several Times. See also Fig. 7.

FIG. 13.

cast before the stems have set, although the contrary has hitherto been generally assumed. Until further tests are made, however, care should be taken to see that the web reinforcement crossing the joint is sufficiently anchored, both above and below, and the surface of the

joint must be clean and not too smooth. As these are conditions not difficult to obtain in practice, there may be now in sight an escape from the worries caused by the occasional necessity of stopping work when the forms are filled only to the bottom of the slab.

Allowable Shearing Stress.—Furthermore, it appears that the stem of a reinforced concrete T-beam may be reinforced so as to be capable of carrying an ultimate shearing stress of at least 888 lb. per sq. in., the highest reached with any of the beams. As the shear was in no case clearly the cause of failure, the really ultimate shearing strength was probably not attained. How much above 888 lb. per sq. in. it really is cannot be stated. The high values of unit shear reached in these tests, together with the fact that the lowest value for the unit shear in any of the beams when the first diagonal crack was observed was in the neighborhood of 185 lb. per sq. in., indicate that the 120 lb. per sq. in. of the Joint Committee's recommendation is low enough, provided the shear reinforcement is adequate. It should be remembered, too, that the concrete of these tests was not of exceptional richness, nor otherwise better than that of ordinary building quality.

DISCUSSION

L. J. Mensch, M. Am. Soc. C. E. (by letter).—These tests present scientific proof of facts which are well known to the experienced worker in reinforced concrete, but are often questioned by engineers and architects. They prove clearly that joints in reinforced concrete construction can be made as strong as a monolithic piece of work, provided they are amply reinforced. It is to be hoped that they will be studied by engineers and architects, and that a study of this kind will prevent such an experience as happened to the writer some years ago in Salt Lake City: He was constructing some girders of 52-ft. span, and the architect compelled the men to finish the girders and slabs in one operation, making them work 20 hours at a stretch and thereby disorganizing the whole gang for a week. Mr. Mensch.

These tests also demonstrate clearly the great influence of stirrups on the strength of reinforced concrete T-beams.

Beams of Type *C*, having the same longitudinal reinforcement as those of Type *A*, but not as many stirrups, show, in the average, 17% less strength than the latter. In the calculation of these stirrups, it will be of interest to apply the rules recommended by the Joint Committee. The area of both legs of a $\frac{5}{16}$-in. round stirrup is 0.153 sq. in., and that of a $\frac{3}{16}$-in. round stirrup is 0.0553 sq. in. The safe shear in these beams will be assumed to be one-eighth of the total load given in Table 2, and amount to 6 350 lb. for beams of Type *A* and 5 380 lb. for beams of Type *C*. The Joint Committee recommends the use of only two-thirds of these values in the formula; they are, therefore, 4 230 lb. in Type *A* and 3 580 lb. in Type *B*; assuming $j\,d = 11$ in., then the spacing of the stirrups for Type *A* equals $\frac{16\,000 \times 0.153 \times 11}{4\,230 \text{ lb.}} = 7$ in., and for Type *B* $\frac{16\,000 \times 0.0553 \times 11}{3\,580 \text{ lb.}} = 3$ in., which is not a very good agreement.

From other tests, the writer has reason to believe that a beam with the same longitudinal reinforcement, but without stirrups, would have failed at a load of about 38 000 lb., or at one corresponding to a safe shear of 4 750 lb.

Assuming that the excess of shear is taken up by the stirrups, then the stress in beams of Type *A* equals $\frac{(6\,350 - 4\,750) \times 5.5}{0.153 \times 11} = 5\,230$ lb. per sq. in., and in beams of Type *B*, $\frac{(5\,380 - 4\,750) \times 5.5}{0.0553 \times 11} = 5\,700$ lb., which is a better agreement, and shows that the stirrups in Type *B* are more highly stressed than those in Type *A*, although these stresses are probably fictitious.

Mr. Mensch. According to the authors, the reinforcing bars were probably cold twisted. In the writer's opinion, however, they certainly were cold twisted, and are equivalent to high-carbon steel bars.

According to the Chicago building ordinance, a stress of 18 000 lb. is allowed for this grade of steel. It is very important to note that Type *A* beams failed at a stress of 58 750 lb., showing a factor of safety of only 3.25, and Type *C* beams at 51 650 lb., showing a factor of safety of only 2.87, in bending. On the other hand, tests made by Professor Schüle, of Zürich, Switzerland, and by others, prove that concrete beams with low end shear, and reinforced with steel of this grade, have failed at a stress in the steel of about 72 000 lb. per sq. in., and some tests with a higher grade of steel, show a stress as high as 140 000 lb. per sq. in. This poor showing of the longitudinal reinforcement, in the case of beams with high end shear, is confirmed by a great number of practically identical tests of monolithic beams made by Professor Bach, of Stuttgart, in 1911. In order to gain a high resistance to shear, therefore, we have to sacrifice a high resistance to bending.

It is also very important to note that the weight of the stirrups in beams of Type *A*, with a stem width of only 4 in. and a total height of only 15 in., is 2.2 lb. per lin. ft. of beam, or 33% of the weight of the longitudinal reinforcement. If tests had been made on beams with a stem width of 16 in. (such beams often occurring in practice), the weight of the stirrups would have amounted to 9 lb. per lin. ft.

Stirrups cost very much more per pound in place than longitudinal reinforcement, very often from 50 to 100% more, and it is clear that it is cheaper to increase the width of the stem than to provide such an excess of stirrups.

The writer cannot agree with the authors' conclusion, that an allowable stress of 125 lb. in shear is low enough. On the contrary, he considers it a limit which should only be adopted very rarely, and then only by experts, as it leads to uneconomical and dangerous designs. This high resistance of end shear could not have been obtained if the anchorages of both straight and bent bars had not been unusually well designed, which is rarely done in practice. Ordinary hooks, 3 or 4 in. long, would never have given these high results.

It is also evident that the substitution of plain, round, high-carbon bars for the twisted bars, would have somewhat delayed the splitting of the beams at the ends.

The tests of beams of Type *B* are certainly surprising. It must be kept in mind, however, that the stress of the longitudinal reinforcement is very low, that the weight of the stirrups is 66% of that of the longitudinal reinforcement, and that their cost is certainly greater

than that of a corresponding increase of width of the web. Professor Bach also made a great number of tests of beams similar to Type *B*, but obtained shearing stresses of only 550 lb. per sq. in. Mr. Mensch.

In the tests made by Professor Bach, and more so in the tests described in this paper, especially for beams of Type *B*, the span is unusually small. It is well known that the results for resistance to shearing and bond of the concrete to the steel rods are more favorable in beams of small span, and the high shear of 880 lb. could never have been obtained in a test beam 20 ft. long.

J. P. SNOW, M. AM. SOC. C. E.—It may be of interest, in this connection, to refer to the action of bolts in experimental timber joints, which are subjected to much the same shearing action as the stirrup rods in question. In the case of plain fished joints, without keys or notches in the timber, failure is always gradual, because the bolts crush into the wood, the bolts assuming a reverse curve and finally parting by tension when the slip becomes great enough to bring the reverse of the bolt at an angle of about 45 degrees. Mr. Snow.

The same action occurs in a wooden treenail or pin in plank lattice bridges. These pins are generally of oak, and the plank is spruce or pine. When slip occurs, the grain of the pin is somewhat crushed and an offset occurs which is somewhat like a reverse curve, but is never true shear.

The shearing strength and bearing value of concrete are so nearly equal that it is doubtful whether, in the authors' experiments, the stirrup rods added anything at all to the shearing strength of the joint between the stem and the slab, in view of the fact that no initial slip occurred.

Roughness of the joints is an undoubted benefit, because the contact area between old and new concrete is thereby greatly increased. The bonding material must possess as much shearing strength as the mortar, or more, therefore steel is a good material for this purpose, but its great excess of strength is wasted as a help in increasing the shearing value of the joint.

D. GUTMAN, M. AM. SOC. C. E.—The speaker would like to know whether it would have had any bearing on the test if the flanges had been made wider? In the average computation for floor-beams or girders, there is much more flange taken in by the contingent floor-slab than is shown. Of course, from abstract reasoning, that would not change the values for the shear; but those things act very queerly sometimes, and the speaker wonders whether the test might not have shown a great variation if there had been at least a foot on each side, because the building codes of various cities allow at least that much. Mr. Gutman.

In practice, the floor-slab has a negative bending moment at the point of support, namely, the beam. That is to say, the flange of the

Mr. Gutman. T-beam would tend to bend downward if the test beam were actually loaded as it is in the building. This force would tend to lift the center portion of the flanges away from the stem.

Inasmuch as there is a greater number of shear bars present than the beam would contain in practice (which extra bars would take care of the effect of the negative bending moment of the slab), is it really fair to compare a separately poured flange and stem and a flange and stem poured in one operation?

Mr. Seelye. ELWYN E. SEELYE, ASSOC. M. AM. SOC. C. E.—The fact that the depth of the beams tested, in proportion to the span, is much greater than that which is ordinarily met in practice, and the fact that the reinforcing rods were either hooked or bent up at the ends, leads the speaker to suspect that the load was carried by arch action rather than by flexure, and, therefore, that the shear along the plane under discussion would be less than the authors estimate it to be. Also, as has been suggested, the presence of the stirrups hooked into the T-flange develops "punching" shear, and hence a higher value may be expected.

The speaker is constrained to take the position of a critic because he feels that it would be a serious matter if the idea got abroad among practical men that the beam and slab do not need to be poured in one operation. It is sufficiently difficult now for the engineer to have his construction joints placed where he wishes them.

Mr. Heiser. ALFRED B. HEISER, JUN. AM. SOC. C. E.—The authors claim to have made a comparative test between smooth and corrugated joints in beams having the flange poured some days later than the web. However, such a comparison is not made, for in none of the specimens tested did the joint fail. All that is proved is that when sufficient stirrups are placed across the joint it will not rupture under the breaking load of the beam.

When beams of this character are used in construction work, there are a number of conditions which do not exist in laboratory practice. In hoisting the beams into position they are jarred and pretty well shaken, thus tending to break the bond between steel and concrete. After the beam is in place and the slab is poured, forms are dropped on it, vibrations of the building shake and jar it, and temperature stresses warp it, all of which tend to destroy the bond between the web and the flange. One year after placement, a beam is likely to have a very imperfect bond between the web and the flange. Any decrease in the grip of the stirrups on the concrete will decrease the frictional resistance along this plane.

In answer to the three objects of the tests, as stated in the paper, it may be said:

1.—That this type of construction is safe for buildings only when sufficient stirrups are placed across the joint.

2.—The tests do not warrant any omission of corrugations, except when using the same proportion of stirrups as in the test. About three times the quantity of stirrups recommended by the Joint Committee were used in these tests. Mr. Heiser.

3.—The authors have not discovered how high a shearing stress the smooth joint will resist, for the joint did not fail.

Beam No. 19 has seven ⅜-in. square stirrups and two ½-in. square bars crossing the joint between the load and the support. The seven stirrups have an area of 1.9684 sq. in. This, resolved through an angle of 45°, in order to resist diagonal tension, is 1.38 sq. in. Adding 0.5 sq. in. (area of toggle bars) to this gives 1.88 sq. in. of steel resisting the tension imposed on the main steel. This gives a unit stress of $\frac{103\,200}{1.88} = 55\,000$ lb. per sq. in., which is approximately the yield point.* This certainly shows that the concrete has not been stressed.

The total tension in the beam is derived by multiplying the area of the tension steel, 2.0 sq. in., by the unit stress in Table 2, 51 600, which gives 103 200 lb.

The authors would do better if they tested a beam with stirrups in the web (sufficient to resist diagonal tension) and with only two or three stirrups across the joint. The beam would then probably fail along the construction joint, thus giving a means of comparison between the two styles of joint, and also values for these joints.

The authors state in the introduction that they are endeavoring to find an economical construction. Stirrups cost money, and if some can be omitted there would be a saving. These tests do not give any means whereby the stress in the stirrups can be measured; a test worth while ought to give the strength of the joint, so that it would be possible to ascertain the number of stirrups necessary to reinforce it. The conclusions drawn by the authors are misleading, and might cause disaster if used by an incompetent designer.

On first reading the paper, one would think that, with the ordinary quantity of stirrups, as recommended by the Joint Committee, a construction joint is permissible in a beam, for the authors do not give their method of calculating these stirrups. As the majority of the members have only time for a first reading, it is thought that this paper might cause laxity in pouring methods.

HENRY G. RAFF, M. AM. SOC. C. E.—The speaker has read this paper with interest, and feels that the authors are entitled to many thanks for publishing the description and results of their experiments. Mr. Raff.

The question of construction joints in concrete and reinforced concrete construction has long been, is, and will continue to be, a

* Taylor and Thompson, p. 38, give 52 500 lb. as the yield point for high-carbon steel.

Mr. Raff. bone of contention between the contractor and the engineer. All efforts to solve this problem in a practical way should, and undoubtedly will, receive hearty endorsement by the members of this Society.

In describing the various types of beams, the paper states that the Type *A* beams were similar to those going into the actual construction. The speaker assumes that these test beams were similar to, but not identical with, those comprising the floor system of the buildings.

The paper also states that the sections of these test beams, and the reinforcement therein, were designed with the idea of developing high shearing stress in the stems, and, in order to obtain this condition, the various sections and the reinforcement were of more than ordinary size and quantities.

The question that naturally presents itself is, to what extent should actual construction work be governed by the results of these tests? The favorable results shown are very encouraging; however, the speaker would advise caution in the matter, and allow the present method of monolithic construction to prevail until such time as tests on actual construction members prove the advisability of permitting these construction joints in actual practice.

The test beams, *A* and *C*, having the same cross-section and span length (though the quantity of stirrups in *A* is greater than in *C*), permit of direct comparison, and lead to the conclusion that the stirrups assist the construction joint in taking shear. The test beams, *A* and *B*, or *C* and *B*, being of unequal spans, do not permit of this direct comparison, but, eliminating the consideration of the different quantity of stirrups therein, an interesting comparison may be made with reference to span length; likewise, eliminating the consideration of the unequal length of spans, a comparison may be made with respect to the quantity of stirrups contained in them.

Table 3 has been derived by averaging the various groups of "results of tests" given in Table 2, and may prove to be of interest.

TABLE 3.

No.	Type.	Joint.	Total load.	Stress on concrete, in pounds per square inch.	Total shear.	Shear area at construction joint, in square inches.	Shear per square inch.	Total stirrup area at joint, in square inches.	Percentage of stirrup area, in terms of shear area.	Span of beams, in inches.
1...	*A*	Smooth..........	50 700	2 885	230 000	400	575	2.76	0.69	100
2...	*A*	Corrugated	47 500	2 738	217 200	400	543	2.76	0.69	100
3...	*A*	Monolithic.	49 800	2 815	225 200	400	563	2.76	0.69	100
4...	*B*	Smooth..........	51 800	2 573	214 928	304	707	5.06	1.66	76
5...	*B*	Corrugated	60 600	2 980	250 192	304	823	5.06	1.66	76
6...	*B*	Monolithic.	55 550	2 695	227 392	304	748	5.06	1.66	76
7...	*C*	Smooth..........	43 100	2 665	201 600	400	504	0.99	0.25	100
8...	*C*	Monolithic.......	46 350	2 720	210 800	400	527	0.99	0.25	100

Assuming the stirrups to be capable of taking horizontal shear to the amount of the product of their normal projection on the plane of the construction joint by the ultimate (unit) crushing strength of the concrete (as determined by the average of the stresses given in Table 3), we have a value in horizontal shear for one $\frac{3}{16}$-in. stirrup of $(2\,741 \times 0.1875) = 514$ lb.; a value in horizontal shear for one $\frac{5}{16}$-in. stirrup of $(2\,741 \times 0.3125) = 857$ lb.; and for one $\frac{3}{8}$-in. stirrup a value of $(2\,741 \times 0.375) = 1\,028$ lb. There were 36 stirrups in each beam, hence the total value in horizontal shear of the $\frac{3}{16}$-in. stirrups per beam is 18 504 lb., of the $\frac{5}{16}$-in. stirrups per beam the value is 30 852 lb., and of the $\frac{3}{8}$-in. stirrups per beam the value is 37 008 lb. Mr. Raff.

Table 4 gives the results obtained by deducting these assumed values of the stirrups in horizontal shear from the total shear on the construction joints of the various beams, subtracting from the gross area of these joints the area of their respective stirrups, and dividing the net shear in concrete by the net concrete area of the construction joints.

TABLE 4.

No.	Type.	Joint.	Total shear at construction joint.	Total shear in stirrups.	Total net shear in concrete.	Net concrete area, in square inches.	Total stirrup area, in square inches.	Shear in concrete per square inch.	Span of beams, in inches.
I	*A*	Smooth............	230 000	30 852	199 148	397.24	2.76	501	100
II	*A*	Corrugated	217 200	30 852	186 348	397.24	2.76	469	100
III	*A*	Monolithic	225 200	30 852	194 348	397.24	2.76	490	100
IV	*B*	Smooth..............	214 928	37 008	177 920	298.96	5.06	595	76
V	*B*	Corrugated	250 192	37 008	213 184	298.96	5.06	713	76
VI	*B*	Monolithic	227 392	37 008	190 384	298.96	5.06	636	76
VII	*C*	Smooth..............	201 600	18 504	183 096	399.01	0.99	459	100
VIII	*C*	Monolithic	210 800	18 504	192 296	399.01	0.99	483	100

TABLE 5.

Comparison of beams by numbers.	Unit shear from Table 3.	Unit shear from Table 3.	Unit shear from Table 4.	Unit shear from Table 4.	Percentage of low shear in terms of high shear.
1 and 7..........	1 at 575 lb.	7 at 504 lb.	....	....	88
I and VII.........	....	....	I at 501 lb.	VII at 459 lb.	92
3 and 8............	3 at 563 lb.	8 at 527 lb.	....	....	93
III and VIII.....	....	....	III at 490 lb.	VIII at 483 lb.	100
4 and 1............	4 at 707 lb.	1 at 575 lb.	....	....	81
IV and I..........	....	....	IV at 595 lb.	I at 501 lb.	84
4 and 7............	4 at 707 lb.	7 at 504 lb.	....	....	71
IV and VII.......	....	....	IV at 595 lb.	VII at 459 lb.	77
5 and 2............	5 at 823 lb.	2 at 543 lb.	....	....	66
V and II.	....	....	V at 713 lb.	II at 469 lb.	66
6 and 3............	6 at 748 lb.	3 at 563 lb.	....	....	75
VI and III........	....	....	VI at 636 lb.	III at 490 lb.	77
6 and 8............	6 at 748 lb.	8 at 527 lb.	....	....	70
VI and VIII......	....	....	VI at 636 lb.	VIII at 483 lb	76

Mr. Raff. Table 5 will undoubtedly prove interesting because of the extraordinary results shown in the column headed "Percentage of low shear in terms of high shear." The speaker feels that, irrespective of the various causes contributing to these erratic results, they should receive profound study by the engineer who contemplates the use of construction joints in T-beams similar to those under discussion.

Mr. Doyen. G. E. DOYEN, ESQ.—According to the usual method of calculating stirrups, there seem to be too many in these beams, particularly in those of Type *B*. The usual method of calculating assumes 120 lb. per sq. in., or, at most, 150 lb. per sq. in., on the net section as the allowable shear, and that one-third of this is carried by the concrete and the remainder by the stirrups and bent-up bars. Where there are no bent-up bars, as in this case, the speaker would calculate the stirrups as follows: Width of beam multiplied by unit shear carried by stirrups = 4 in. $\times$ 100 lb. = 400 lb.; value of two $\frac{3}{8}$-in. square bars at 16 000 lb. per sq. in. = 4 500 lb.; the stirrup spacing would then be $\frac{4\,500}{400}$ = $11\frac{1}{4}$ in., or practically three times that actually used. This spacing is too wide for the beam in question, and smaller steel should be used.

The result of using so much stirrup steel is that the web and flange of the beam are so thoroughly tied together that there is no chance for a shear crack to form between them. If the stirrups act in tension, as assumed, and as they undoubtedly do after the web of the beam has begun to crack, there is a direct pull by each stirrup on the flange of the beam, which amounts to quite a considerable force. This pull brings into play a frictional resistance between the two parts of the beam, which acts in addition to any adhesion of the concrete surfaces. Aside from this factor, there can be no doubt that the stirrups prevent any such lifting up of the flange as would be necessary in order that it might slip over the rough surface of the top of the web.

In short, the authors have attempted to find how high a shearing stress a smooth joint between the web and flange of a concrete beam would carry, but have only discovered that, by using a quantity of stirrup steel greatly in excess of what would be used in practical design, it is possible to tie the two parts of the beam together so thoroughly that they are able to obtain a very high unit shear. The tests do not show how high a shear such a joint would stand in a beam designed to meet practical conditions.

Mr. Wiggin. THOMAS H. WIGGIN, M. AM. SOC. C. E.—The speaker is not in accord with those who have suggested the absence of shear on the joints tested; he believes it must be conceded that there is shear on these joints, and that the vertical shear rods in themselves cannot take it. The rods are not placed in the right position to take the horizontal shear, being at right angles to that stress; and the bearing

area which they have on the concrete above and below would be inconsiderable compared with the shear to be resisted. The shear must be carried by the strength which has been developed in the construction joint, explained by excellent adhesion or by some other phenomena, which the discussions thus far have not disclosed. Mr. Wiggin.

With respect to the adhesion as bearing on the integrity of the concrete across the joints, that, of course, is a question of methods of construction. Strong adhesion can easily be obtained in an experimental way, which might not be carried on with such uniform success on a larger scale; for example, the dirt on the surface can all be removed, and with certainty, though it is understood that it was not done in this case. Dirt can be prevented from getting on the concrete and a higher degree of workmanship can be secured than would be generally possible on a large scale.

It has often been shown in experiments that beams of plain concrete can be made in two operations, using rich mortar at the joint, and that these will break elsewhere than at the joint; yet, in construction, if any cracking occurs in concrete, it is usually at the joints and not elsewhere. The speaker has noticed this repeatedly in various shrinkage phenomena, and in testing concrete pipes by water pressure.

One may concede that the methods used were fairly representative of commercial work, and still find an explanation for the strength developed. In tests of what is sometimes called "true" shear, it is very seldom that the breaking stress is less than 700 or 800 lb. It frequently runs up to 1 000 or 1 100 lb., and results greater than 2 000 lb. have been obtained in punching shear on plates with edges bound by reinforcement.* Now, what is there that distinguishes punching shear from ordinary shear developed in beams resting on supports? The nearness of the load to the point of support and the binding force supplied by the grips—and particularly by the steel in the tests, above mentioned, of punching shear on concrete plates with edges reinforced—probably both contribute to the high results; and it is not difficult to conceive that the stirrup steel produced a similar effect in the tests under discussion. The action conceived of is not that suggested by one speaker—that the tension in the stirrup steel causes a friction between the two concrete surfaces assumed to be plane—for tension would cause the stirrups to stretch slightly, thus taking the concrete out of bearing, and precluding friction of the ordinary kind, such as would exist between plane surfaces held together. As no surface under the microscope is smooth (and particularly a concrete surface), it is entirely conceivable that, although the stirrup rods may stretch to a slight extent, they would still hold the teeth of the surfaces, so to speak, in mesh, and thus cause an action

* *Bulletin No. 8*, University of Illinois Engineering Experiment Station.

Mr. Wiggin. similar to punching shear, though not as effective on account of the general absence of pieces of the coarse aggregate across the plane of shear. A minute distortion would have to take place before such action could occur, but it would be very minute with the quantity of stirrup steel used in the beams under discussion.

The speaker offers this as a purely off-hand and unstudied speculation, although it is perfectly certain that, with good workmanship, a joint would frequently be made that would be just as strong as the original concrete. If the explanation suggested has any weight, it is true, of course, that a large number of stirrups would be more effective than a small number, so that there may be something in the criticism in the discussion that the relatively large quantity of stirrup steel does have an effect on the results. The speaker thinks more investigation would be wise before applying the method described by the authors to ordinary designs of reinforced concrete over large floor areas.

Mr. Saurbrey. ALEXIS SAURBREY, ASSOC. M. AM. SOC. C. E. (by letter).—In order to appreciate fully the importance of the tests under discussion, it seems necessary to take into account the earlier history of the principles involved. As a commercial proposition, the unit system referred to by the authors, was first used on a three-story building erected for the American Piano Company, at East Rochester, N. Y., in 1905, which was designed and constructed by the Ransome and Smith Company. This system, known as the "Ransome Unit System," has since been used on a number of one- and two-story buildings, and during the last three or four years, also on buildings from four to six stories high, notably those of the United Shoe Machinery Company, at Beverly, Mass.

In view of the fact that all these buildings were designed along usual lines (except for the feature of the joint between slab and stem and other features connected with unit construction), and that they had been submitted to the test of actual service for several years, without developing any signs of weakness, there was really not room for much doubt as to the outcome of the tests described by the authors, that is, it was pretty well known that the joint would not prove detrimental. It was hoped, however, that the tests would show conclusively the limit at which the joint between slab and stem would separate, so that definite data might be available for future design of the stem and stirrups. While the test schedule was under preparation, the writer, then Chief Engineer of the Ransome Engineering Company, realized that Types *A* and *B* were equipped with an excess of stirrup reinforcement, and he therefore suggested Type *C*, with much lighter stirrups. To compensate for the resulting lower general strength of the beams, the arrangement of the tension rods was reversed, so that the heavier rods were placed on top, and trussed, as shown by the diagrams.

Mr. Saurbrey.

In addition to the experience gathered by the Ransome organization in regard to the effect of joints between slab and stem, it was known that the Hennebique engineers had used such joints, in monolithic construction, both in Europe and in this country. The writer is unable to state what the prevailing practice in Europe is at the present time, but, in this country, such joints are specifically prohibited by the building codes of most large cities. Why this should be, is hard to tell, as there seem to be no test data or other experience showing the joint to be dangerous in any way, except, of course, when the stirrups are inadequate, in which case the construction of T-beam sections would be dangerous without, as well as with, the joint. The writer has had occasion to discuss this question with a number of engineers and contractors, and opinions seemed to be fairly evenly balanced for and against the use of the joint.

Theoretically, it has been found possible to determine by calculation the proper number of stirrups for beams having a joint between slab and stem; and it is also possible to determine the maximum permissible spacing of the stirrups. These questions are dependent, for their solution, on the view taken of the action of the stirrups. The writer believes that the stirrups are stressed in tension, and not in shear, and that there is no shear in a properly designed, reinforced concrete beam after the stage of initial loading has been passed (when the steel is stressed above, say, 5 000 lb. per sq. in.), not even in the concrete; the views and arguments leading to this belief have been published elsewhere.* The tests under discussion seem to substantiate these views. (See, for instance, Fig. 11.) The flaking of the surface of the beam is plainly discernible; the concrete of the stem has lost its cohesion to such extent that large pieces can be picked out with the fingers. In other words, the beam is nothing but a pile of loose pieces, held in place by the horizontal and vertical rods, and by the dove-tailing of the individual pieces. It is impossible to understand how concrete in this stage can transmit "shear stresses," which, it is known, involve tension stresses, and yet, these beams carried considerable additional load before they broke down entirely.

Under these circumstances, it seems futile to discuss the calculated intensity of the shear stresses. To the writer it seems that the steel, whether stirrups or main tension rods, carries the tension which undoubtedly occurs in both horizontal and vertical directions, and the loose filling of destroyed concrete carries the corresponding compressive stresses. When the destruction of the concrete reaches the end of the beam, the anchorage of the tension rods is finally destroyed, and the beam naturally fails. The writer mentions this with confidence, as he was present at most of these tests. He takes pleasure in acknowledging

* "Reinforced Concrete Buildings," Ransome and Saurbrey, 1912, Chapter VII.

Mr. Saurbrey. the unusual care exercised in placing the beams, measuring deflections, increasing the loads, in short, in everything pertaining to the testing. If any criticism could be offered, it would extend only to the neglect of moistening the beams from day to day, during the hardening period, and to the absence of control tests of compression specimens. In regard to the first point, the writer sees therein the cause of the observed fact that the beams 40 days old, and those 90 days old, had practically the same strength, for it seems to be well established that concrete will not increase much in strength after a certain period, unless it is kept in a moist condition. In regard to the second point, compression specimens would have been useful for comparison with other series of tests, and, therefore, would have enhanced the value of the tests as a whole. For the particular purpose of the tests under consideration, these objections are, of course, of no importance.

The writer agrees with the authors' statement, namely, that "where adequate web reinforcement is provided with sufficient anchorage at the top, the slab need not be cast before the stems have set." That this is true for beams like those tested, is beyond doubt. That it would also be true for beams constructed along different lines may be inferred from the fact that both the flange and the reinforcement along the bottom of the beams, were purposely made unusually heavy, so that the stresses along the joint must of necessity have been abnormally high, whatever view one chooses to take of the action of the stirrups. Mr. Gutman raises the point as to the effect of a wider and thinner flange. It would probably be difficult to examine this point in the laboratory, where the width of the flange is determined by the construction of the testing machine. However, the anchorage of the stirrups in the slab is a factor of the greatest importance, and this detail is much more difficult in a thin slab than in one of generous dimensions. Otherwise, the writer does not see that the width of the flange enters into the problem.

In order to exhaust completely the possibilities of construction, many more tests will have to be made, and on account of the very real importance of the question, both in monolithic and unit methods of construction, it is hoped that this will be done. It is in itself a considerable task to make and test 28 large-sized beams, as was done by the authors, and the beams tested were naturally constructed to conform, as closely as possible, to the problems encountered in the two buildings then under construction in Boston, and since completed. As far as the writer knows, no structural defects have developed in these two buildings, nor has the joint between slab and stem caused trouble in any of the buildings erected before these tests were made. It is to be hoped that the building authorities everywhere will recognize the importance of the information gathered by the authors, and revise their laws accordingly.

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LEWIS J. JOHNSON, M. AM. SOC. C. E., and JOHN R. NICHOLS, Assoc. M. AM. SOC. C. E. (by letter).—The discussion has raised few points which were not anticipated in the paper or which seem to need more elaborate treatment than was given there, but, in the light of what has been said, the following is believed to be in place.

First, the writers wish to repeat that the tests described were undertaken solely for the purpose of investigating the construction joint between the slab and the stem of concrete T-beams, when these two parts are cast at different times. This general purpose divided itself naturally into three specific questions, which are stated early in the paper. The joints were expected to fail, if they failed at all, in shear; and in order to give them as severe a test as possible, it was deemed necessary to subject them to a relatively high shearing stress. To this end the spans were made short and the stems comparatively deep and narrow. To prevent premature failure by diagonal tension in the stem, stirrups were liberally provided.

The question of economy in the construction of these beams was of no importance, and has no place in the discussion of the tests. From the point of view of good design for actual building construction, there were features in the test beams much worse than lack of economy in the quantity of web steel. Elimination of these features, however, would have destroyed the value of the beams for the tests in hand, and, under the circumstances, would have served no useful purpose.

The foregoing remarks apply to beams of all three types. Referring now to Type *B* beams, for the reason that these were designed primarily to develop the highest possible shearing stress, and, having no inclined steel, are simpler to analyze, their design will be explained in some detail. According to the ordinary assumption of straight-line distribution of compression and absence of horizontal tension in the concrete, the beams were designed so that, under a load of 19 700 lb., the following unit stresses would be developed:

Tension in main steel..........	16 000 lb. per sq. in.
Tension in stirrups............	15 240 " " "
Compression in flange..........	962 " " "
Adhesion	133 " " "
Shear on joint.................	266 " " "

The stirrups were designed to carry the whole shear, as follows:

Cross-section of two $\frac{3}{8}$-in. bars = 0.281 sq. in.

$$\text{Shear on cross-section of beam} = \frac{19\,700}{2} = 9\,850 \text{ lb.}$$

The distance from the center of the steel to the center of compression is computed to be 9.2 in.

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With 4-in. spacing, the stress is computed by the usual method to be:

$$\frac{9\,850 \times 4}{0.281 \times 9.2} = 15\,240 \text{ lb. per sq. in.}$$

This method of determining stirrup spacing undoubtedly calls for more steel than the methods used by many engineers, more than is recommended by the Joint Committee, and perhaps more than is required by good practice in concrete structures; but these Type *B* beams, as previously stated, were designed for a special test which required thorough precautions against failure by diagonal tension. The method used was therefore plainly justified.

The question that next arises is to what extent, if any, the stirrups aided the joint in carrying its shear load, or, in other words, relieved the shearing stress on the joint.

Mr. Snow has already pointed out the analogous case of two pieces of timber fastened together by bolts in shear, and the impossibility of causing large shearing stress in the bolts without considerable yielding and slipping of the timbers. For a closer analogy with the test beams, suppose two boards lapped at their ends, glued together, fastened together by bolts through the glued surface, and subjected to tension so that the glued joint is stressed in shear. It is obvious that the bolts, however tightly they fit, cannot be stressed appreciably until the glue has failed. The glue in shear is a much more rigid connection than the bolts in combined shear and bearing and, consequently, takes the brunt of the pull tending to slide the boards. In the tests of these beams, no sign whatever of slipping of the joint was observed, although, of course, it was this particular phenomenon that was watched for with the greatest care. Accordingly, by the same reasoning, the concrete joint, so long as it remained intact, being far more rigid than the steel stirrups in combined shear and bearing, consequently withstood the bulk of the shear to which it was subjected. The writers venture the opinion that the shearing stress on the steel at the joint was not substantially greater than that on the adjacent concrete.

Suppose, however, for argument's sake, that the shear on the stirrups actually did rise to the greatest value that could be borne without either crushing the concrete against the side of the stirrup on each side of the joint, or bending the stirrup. It is possible to estimate roughly the amount of this shear, as follows: Assume that the portion of a stirrup above the joint is subject to the shear, S, at the joint, and horizontal pressures, as indicated in Fig. 14. Call M the flexure in the stirrup at a distance a (where the horizontal pressure vanishes) from the joint.

The resultant R of the pressure of the concrete against the stirrup is assumed to be equal to S. From this it follows (if we ignore the adhesion forces on the stirrup in the space, a) that M is the maximum

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flexure in the stirrup. If the point of contraflexure in the stirrup comes at the joint, we have:

$$M = S\frac{a}{3} = R\frac{a}{3}$$

$$\text{whence } a = \frac{3\,M}{S}.$$

If the maximum pressure against the side of the $\frac{3}{8}$-in. stirrup is 3 000 lb. per sq. in., we may write:

$$S = R = \frac{3\,000 \times 0.375}{2}\,a.$$

Substituting the value of a from above:

$$S = \frac{3\,M}{S} \times \frac{3\,000 \times 0.375}{2}$$

$$S^2 = 4\,500 \times 0.375\,M.$$

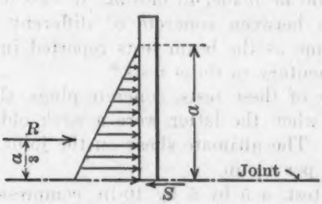

FIG. 14.

The flexure in a $\frac{3}{8}$-in. square bar for a maximum fiber stress of 48 000 lb. per sq. in. cannot exceed 422 in-lb. Substituting this value in the above equation, we find:

$$S^2 = 712\,000$$

$$S = 842 \text{ lb.}$$

For each bar of a stirrup there are 8 sq. in. of concrete at the joint. If the relative rigidities of the concrete and the steel were such as to cause a shear of 842 lb. in each stirrup bar at the joint, the relief afforded to the concrete would be only about 105 lb. per sq. in.; but, even if twice this amount (making excessive allowance to offset having ignored the adhesion forces on the stirrup), or 210 lb. per sq. in., were subtracted from the average shear, computed and given in Table 2 for the smooth joint, Type B beams, 713 lb. per sq. in. (which, be it remembered, was not the ultimate strength, but only the stress at failure from other causes), that would leave 503 lb. per sq. in. for what the joint itself actually carried without sign of failure from this

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cause, and the general conclusion drawn from the tests would stand confirmed as stated:

"That a smooth or unroughened joint, constructed in the same manner as the joints tested in a beam suitably reinforced for shear (or diagonal tension), is capable of transmitting with ample factor of safety any shear that could safely be permitted in a monolithic beam of the same cross-section and having the same reinforcement."

Although it is probably true that the quantity of steel in the stirrups affects in some degree the strength of the joint, aside from any doweling action, in that they serve by direct tension to hold the surfaces of the joint in close contact, the writers are of the opinion that a large variation in the quantity of steel would affect the strength of the joint but slightly. It matters little in this respect whether the beam contains, for instance, twelve stirrups stressed to 8 000 lb. per sq. in., or six stirrups stressed to 16 000 lb. per sq. in. It is unquestionably desirable, however, that some steel should be provided crossing the joint, either in the shape of stirrups or otherwise.

Reference should be made, in closing, to two tests on the shearing strength of joints between concrete of different ages, conducted at about the same time as the beam tests reported in the paper, and regarded as supplementary to those tests.*

Briefly, for one of these tests, concrete plugs, slightly tapered, cast in concrete rings when the latter were a week old, were later pushed out, big end first. The ultimate shear on the joint averaged, for three specimens, 665 lb. per sq. in.

For the other test, a 5 by 5 by 10-in. compression prism was cast with an oblique construction joint. The concrete below the joint was 4 days old when the concrete above was cast against it. The compression load was applied at the ends of two such specimens, and they failed in shear at the joint. The average ultimate shearing stress on the joint planes was 558 lb. per sq. in. In neither of these tests did any steel cross the joints.

Although the supplementary tests were hardly numerous enough by themselves to warrant any sweeping conclusions, they corroborated the evidence secured by the beam tests.

All the tests taken together point clearly to the conclusion that a carefully constructed joint of the kind tested, when it is not in tension, or when steel rods in reasonable quantity, sufficiently anchored, cross it at right angles, need not be regarded as a weakness in a structure.

* The details and results of these tests are recorded in the *Harvard Engineering Journal*, of April, 1913 (Vol. XII, No. 1), and a summary is given in *Engineering Record*, of May 3d, 1913.

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Paper No. 1308

STORAGE TO BE PROVIDED IN IMPOUNDING RESERVOIRS FOR MUNICIPAL WATER SUPPLY.*

By Allen Hazen, M. Am. Soc. C. E.

With Discussion by Messrs. Hiram F. Mills, T. U. Taylor, F. B. Marsh, L. J. Le Conte, E. P. Goodrich, James L. Tighe, H. T. Cory, and Allen Hazen.

There is undoubtedly a definite relation between the storage provided in an impounding reservoir on any stream and the quantity of water which can be supplied continuously by it. The relation, however, is a complex one, and our knowledge of its character is limited. The following study is made to see how far it may be possible to separate this complex relation into parts, some of them being of such a nature that they may be studied separately with definite results, and afterward to treat all the remaining variations on the basis of probabilities, using all data from a number of different streams; and to study them in comparison with the normal law of error.

Among the elements that can be studied separately are the following:

1. The size of the catchment area.

2. The mean annual run-off per square mile.

3. The portion of water area and the loss by evaporation from it. This relation is a complex one, and data for determining it are less adequate than could be desired. Nevertheless, some approximations can be reached.

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*Presented at the meeting of December 17th, 1913.

4. The natural storage in lakes, or in deposits of sand and gravel and other pervious materials. Only approximate results for natural storage can be reached, but as these are found to have a great influence on the required storage, especially at relatively low rates of draft, they must be considered.

5. Regularity in annual flow. Some streams have comparatively regular flows; in others the variation is much greater. This difference in regularity of flow can be taken into account by finding a coefficient determined from the record of each stream, and bringing this into the statement in such a way that variations from the normal are stated in terms of the "standard variation". In this way, records of streams having more regular flows and those having less regular flows may be compared with reference to other matters.

For the present, all remaining elements of variations of flow of every description will be thrown into one group and studied in connection with the normal law of error.

In general, it may be said that, as more information is secured as to any part of the whole problem, it tends to reduce unexplained variations, and to permit more accurate analysis of other parts. The whole study, therefore, becomes one of successive approximations. The results herein contained are to be taken as only one step in the development. It is to be expected that further study will show reasons for deviations, the causes of which are not now apparent, and will ultimately lead to more certain and accurate knowledge of the whole subject.

The methods developed in connection with the study of the normal law of error are well suited to an investigation of this kind. There is a presumption that they may be applied, growing out of experience with other kinds of data, but the presumption is not so strong that it is to be accepted without a careful study of the best flow data available, to see how close the agreement really is, and how far it may be depended on.

The first requisite to a successful study of probabilities is to have ample data. There are only a few cases where the records of stream flow cover a longer period than 25 years, and the longest record here used, that of the Croton River, covers only 45 years. No one of these records, taken by itself, is sufficient to serve as an adequate

basis for forming judgment of the probability of the recurrence of conditions which are only to be expected at intervals longer than are covered by these records.

In order to form a judgment of such conditions, we may either resort to the normal law of error, and assume that, if it applies reasonably to the data for short periods, it will also apply to longer ones; or we may combine the records of several streams, after eliminating all the elements that can be separated and eliminated, thus forming a single series containing the elements of unexplained variation reduced as nearly as possible to a common basis. In this way a series of results is built up which may be taken as representing the unexplained variations in the flow of a single stream for hundreds of years.

Such an artificial record is open to some objections. All the records of which it is composed were obtained in the same general period of time, and any changes in climatic conditions that might take place in a long term of years are not reflected by it. Nevertheless, such an artificial series seems to be the best means now available for finding approximately some of the relations between flow and storage.

Study Divided into Two Parts.—An impounding reservoir serves two purposes. These run into each other, more or less, and overlap, but they are nevertheless reasonably distinct, and can be best considered separately.

The first is to balance the fluctuations in flow during the seasons of one year. That is, to hold the flood flows of the winter and spring and make them available for maintaining the service during dry periods in the following summer and fall. This will be called the monthly storage.

The second is to hold the surplus water of wet years and make it available in the dryer years that follow. This will be called the annual storage.

These matters will be taken up for discussion separately, and afterward the results will be combined in a single statement.

Approximate Methods Used.—In all hydraulic data the probable error of measurement is considerable. There is, therefore, no justification for the application of extreme refinements in methods of

calculation. With this in mind, slide-rule calculations have been used. In most cases, available records are given as monthly averages, and each month has been taken as one-twelfth of the year, regardless of the number of days it actually contained. In a few cases, daily or weekly records are available, and a brief investigation has been made as to the probable error involved by the use of monthly average figures instead of daily ones. Some changes of method have been made during the course of the work, and minor discrepancies resulting therefrom (too small to be significant) have not always been corrected. Where records from several streams are to be averaged, a weighted average is used in which each is given a value in proportion to the length of the records from which it is obtained.

Land Area Only as Basis.—In the case of many of the streams included in the following study, all or nearly all the catchment area is land; that is to say, it is not occupied by the water surface of reservoirs. In other cases, reservoirs or lakes have occupied a certain percentage of the area, and this percentage has gradually increased during the period covered by some of the records. The published records of flow are based on the whole area, including water surface. For the purpose of this study, the method of taking only land area as a starting point, as suggested by Frederic P. Stearns, Past-President, Am. Soc. C. E., has been followed. The published figures are revised by dividing the run-off per square mile from the total area, by one less the proportion of water area.

DATA USED.

The first step is to reduce existing data to a land-area basis, and make a tabular statement showing the average flow per square mile for each catchment area for each month for the whole period covered by the observations. The records of flow of the following streams have been used:

Sudbury River.—This is a part of the Boston water supply, with a catchment area of 75 sq. miles. The records cover the period from 1875 to 1896, inclusive, during which time the water area ranged from 2 to 4 per cent. The more recent records of the Sudbury River are not used, because, beginning with 1897, the water for Boston from the Wachusett catchment area was drawn through the Sudbury system, and the Sudbury figures are obtained as the difference between

the measurements of the water entering and leaving. As the quantities passing through are much larger than those originating in the area, the measurement is not believed to be sufficiently accurate to justify the use of the records since this condition has existed. The Sudbury catchment area is rolling, inhabited, and cultivated, and has much sand and gravel.

Wachusett Reservoir (South Fork of the Nashua River).—This is also a part of the Boston water supply. The records cover the period from 1897 to 1911, inclusive. The catchment area of 118 sq. miles is somewhat more hilly and higher in elevation than the Sudbury. It also contains a large quantity of sand and gravel. The water surface has ranged from 2 to 7 per cent.

Croton River.—This is a part of the New York water supply, with a catchment area of 339 sq. miles at the Old Croton Dam, increased in 1906 to 360 sq. miles at the New Croton Dam. The records cover the period from 1868 to 1912, inclusive, during which time the water area has ranged from 2 to 5 per cent. The catchment area is rolling, is cultivated to a considerable extent, and has a large quantity of sand and gravel.

Manhan River.—This has a small catchment area of 13 sq. miles, forming part of the water supply of Holyoke, Mass. The measurements were made by weirs, with unusual care, and cover the period from 1897 to 1910, inclusive. The area is rolling, and the quantity of sand and gravel is large.

Catskill Streams.—These include the Esopus, 378 sq. miles; Schoharie, 240 sq. miles; and Rondout Creek, 105 sq. miles. The gaugings were made by the City of New York, and cover a relatively short period. The catchment areas are steep and mountainous, and largely wooded. The soil is generally impervious, and there is little sand and gravel. The records are not of sufficient length to cover the dryest periods, but they are useful as indicating the ordinary conditions of storage required in catchment areas differing radically in physical characteristics from those previously mentioned.

Pequannock River.—This furnishes the water supply for Newark, N. J. It has a catchment area of 62 sq. miles of hilly, almost mountainous country, cultivated to a moderate extent, but largely covered with second-growth forest. The record covers the period from

1892 to 1911, inclusive, and is made up of Venturi measurements of the water taken out of the catchment area for use, and weir measurements of the water washing over the intake dam. The water area is about 4 per cent.

Philadelphia Streams.—Three streams, Perkiomen Creek, 152 sq. miles; Neshaminy Creek, 139 sq. miles; and Tohickon Creek, 102 sq. miles, were proposed many years ago as sources of additional water supply for Philadelphia. They were not used as proposed, but careful gaugings, extending over a period of 25 years, are available. The country is hilly and partly cultivated.

Gunpowder River.—This furnishes the water supply for Baltimore, Md. The catchment area is 308 sq. miles of hilly, rolling country, under a good state of cultivation. The hills are high and steep, but there is a deep cover of fine-grained micaceous sandy material on a large part of the area, and this serves to store a large quantity of water, so that the ground-water flow of this stream is larger relatively than that of any other stream considered. The records are made up of the quantity of water drawn for use by the City of Baltimore, and that flowing over the intake dam, calculated from the records of gauge heights covering the period from 1883 to 1911, inclusive. In considering these records, in their report on the water supply, Messrs. Freeman and Stearns reduced them slightly because they believed that the coefficient to be used in the weir formula, for the crest of the dam as it existed, was lower than had been assumed in making up the quantities used by the Water Department. In this calculation the records are used without correction.

Merrimac River.—The Merrimac River drains an area of 4 634 sq. miles. The record of the flow at the Lawrence Dam has been kept by the Essex Company. Lake Winnepesaukee and many smaller lakes, comprising 2.6% of the total catchment area, are included in this area, and storage in them, either natural or with the aid of the control works at the outlets of some of the lakes, is an element in maintaining the regularity of the flows. No correction for water area has been made in the Merrimac flows.

Hudson River.—A record of the flow has been kept at Mechanicsville, where the catchment area is 4 500 sq. miles. As with the Merrimac, there are numerous lakes on the catchment area, and natural

storage is an element in maintaining the flows. No correction has been applied for water area.

Colorado Streams.—The gaugings of three streams used by the Denver Union Water Company are available:

1. Bear Creek; record, 1900 to 1911, inclusive, and a catchment area of 172 sq. miles;

2. South Fork of the South Platte River; record, 1900 to 1911, inclusive, and a catchment area of 1 796 sq. miles;

3. South Platte River at Platte Canyon; record, 1903 to 1911, inclusive, and a catchment area of 2 688 sq. miles.

These streams were included as typical ones in a dry country where the variation in flow from year to year is much greater than in any of those previously mentioned. The seasonal fluctuation in flow also depends on different climatic conditions, especially on the accumulation of snow in the high mountains, and its melting in summer. These streams were included in the study in order to see how far the same methods of calculation would be applicable to them, and with the idea that they would be included in summaries with the eastern streams only if it became apparent that doing so would not modify materially the standards reached. Including these western streams has the important advantage that it tends to broaden the methods used for comparing data, so that they are applicable under a greater variety of conditions.

Not all the sources mentioned were used for all parts of the following study, but only such records as seemed most appropriate in relation to each point taken up in turn.

Additional Records.—It would be possible, by using the records of the United States Geological Survey, greatly to extend the investigation. In the first study it is thought best to use only a limited number of selected data. It is by no means an easy matter to gauge streams accurately, especially under winter conditions in northern climates, with large quantities of ice and anchor ice in the water. It is necessary, therefore, to use these records with caution, because such errors may exist in some of them. Long-term records are obviously more useful than those of short term, and the latter have been considered only in the case of a few streams representing types of catchment area not otherwise included.

Calculation of Storage.—Calculations were made for each year for each of the streams investigated in order to determine the storage required to maintain each of a number of assumed rates of draft. For most of the streams, for which the flows were given in gallons per square mile, these calculations were made for rates of draft of 100 000, 200 000, 400 000, 600 000, 800 000, and 1 000 000 gal. daily per sq. mile of land area, or for so many of them as required storage and could be supported. In the case of streams where the records were in cubic feet per second, rates of draft were taken giving about the same general range.

Basis of Stating Storage.—There are several ways in which the storage may be stated: In relation to the land area, in terms of the mean annual run-off, or in terms of the proposed maintainable draft.

When storage is based on the tributary area, it may be stated as inches of run-off or millions of gallons per square mile of tributary area. These and similar terms bear fixed relations to each other. Thus 1 in. of run-off is always equal to 17 400 000 gal. per sq. mile. When these units are used, it should be stated whether they relate to the whole area or to the land area only, as either form may be used. When the storage is based on the mean annual flow, the percentage of it required to fill the reservoir from bottom to top is usually given. When the storage is based on the maintainable yield, it is most conveniently stated in days' supply. Thus, if a given reservoir holds 100 000 000 gal., and is used for an output of 2 000 000 gal. per day, 50 days' storage is said to be provided. Each of these three ways of stating storage has advantages for certain purposes, and they are all used in this discussion.

Monthly Storage.

Each Year by Itself.—It is assumed, in the first part of this study, that the reservoir is full each year at the beginning of the summer dry period, and no account is taken of any deficiency that might exist from operations of previous years. The figures thus reached are referred to as those for "each year by itself" or as "monthly storage". A second set of figures is made in which the deficiency at the beginning of any year is carried forward. The figures thus reached are spoken of as "cumulative". The cumulative figures are comparable with those made by Messrs. Stearns, FitzGerald, and Freeman for a zero water area.

At the outset it was supposed that the cumulative figures would be the most important, and those representing each year by itself were carried as a convenient and perhaps useful check. It turned out, however, that the figures for each year by itself could be reduced to more definite and satisfactory order than was possible for the cumulative figures, and in the final study no use is made of the latter, except as a check on the results otherwise reached.

The monthly storage figures, representing the storage for each year by itself, are used as a convenient step in the process of development, and do not always represent the whole quantity of storage required, as they contain no allowance for the annual storage nècessary to hold the excess water of wet years and make it available in dry ones.

The figures for monthly and annual flow for each of the streams, and the computed storage therefrom for several assumed rates of draft, are compiled in Table 1. As the original of this table is very long, and as all the figures used in the subsequent discussion are shown graphically in the diagrams which follow, the table is not presented in full, but only one part of it, containing the records of one stream, in order to show its form.

Plotting Results.—On Fig. 1 are plotted these results for the Wachusett Reservoir. The quantities of storage for each rate of draft are arranged in order of magnitude and plotted on lines equally spaced on the diagram. These lines are not reproduced, but the points plotted show their position. Lines showing the percentage of the total number of years are added as a more convenient basis for further discussion. Lines connecting them approximate in shape to the letter "S", the middle part of the curve being moderately straight, and the curvature near the ends much sharper.

Smooth curves have been drawn to show the most probable curve in case the normal law of error applies to the data. The question as to whether it does apply will not be discussed at this time, but the lines drawn with its aid give a better idea of the normal shape of the curves than can be obtained from the few points that constitute this particular series. If the data were much more numerous, a fair approximation of positions near the ends, that is to say, of the probable storage required to maintain drafts in years so dry that they recur only once in 20 or 50 years, could be obtained.

TABLE 1.—(IN PART.)—AVERAGE RUN-OFF, BY MONTHS, IN THOUSANDS OF GALLONS PER DAY PER SQUARE MILE OF NET LAND AREA, FOR WACHUSETT RESERVOIR.

Year.	Percentage of water area.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Year.
1897	2.2	815	954	2 822	1 669	1 189	1 208	1 474	916	889	248	1 312	2 826	1 281
1898	2.2	1 598	1 672	3 157	2 072	1 421	847	341	1 355	692	1 543	2 219	2 107	1 586
1899	2.2	2 139	1 115	2 838	3 452	882	574	362	242	256	251	440	367	1 075
1900	2.2	814	4 144	3 806	1 616	1 413	591	222	201	131	289	895	1 605	1 292
1901	2.2	531	364	2 779	5 098	2 790	1 007	488	524	327	632	529	3 307	1 541
1902	2.2	1 714	1 432	4 081	2 207	1 054	419	299	304	247	973	650	1 889	1 276
1903	2.4	1 296	2 185	3 507	2 293	583	2 183	639	486	384	705	649	976	1 316
1904	3.6	684	962	3 120	3 095	1 554	791	515	368	512	360	356	457	1 063
1905	4.1	1 320	472	3 132	1 686	464	565	381	335	1 280	383	461	1 061	965
1906	5.1	1 193	1 082	1 960	2 222	1 615	1 248	768	623	292	559	790	836	1 099
1907	6.0	1 551	736	1 805	1 528	1 027	822	356	93	861	1 470	2 702	2 086	1 255
1908	7.0	1 869	1 867	2 357	1 364	1 521	434	237	476	95	170	134	416	910
1909	7.0	637	2 748	2 289	2 604	1 303	680	251	208	224	97	391	578	987
1910	7.0	1 985	1 984	2 839	1 112	655	886	67	200	156	73	381	420	890
1911	7.0	832	672	1 440	1 498	496	377	61	202	195	773	1 113	1 147	733

COMPUTED STORAGE, IN MILLIONS OF GALLONS PER SQUARE MILE OF NET LAND AREA, FOR SEVERAL RATES OF DRAFT. EACH YEAR IS CONSIDERED BY ITSELF, EXCEPT THAT WHERE CUMULATIVE STORAGE IS GREATER, THE FIGURE FOR IT IS GIVEN IN PARENTHESES.

Year.	Computed Storage, in Millions of Gallons per Square Mile of Land Area, for Drafts as Marked.					
	100 000	200 000	400 000	600 000	800 000	1 000 000
1897........	0	0	5	17	29	44
1898........	0	0	2	8	14	25
1899........	0	0	15	52	95	147
1900........	0	2	23	48	78	127
1901........	0	0	2	14	45	75
1902........	0	0	11	35	59	95
1903........	0	0	0.5	10	35	77
1904........	0	0	3	31	68	118
1905........	0	0	3	20	58 (127)	95
1906........	0	0	3	11	30 (189)	65
1907........	0.2	3	11	23	35 (96)	57
1908........	0.2	6	27	68	116	165
1909........	0.1	3	25	57	97 (154)	140
1910........	1.0	9	34	70	112 (206)	173
1911........	1.2	4	23 (59)	51 (153)	82 (297)	119

With data no more numerous than those in the record shown by Fig. 1, the method is a clumsy one, and cannot be expected to yield close results. It may be pointed out, however, that the method

which has been most commonly used for estimating storage has been that of considering only the highest term in the storage series, and that no consideration has usually been given to the other terms. Under these conditions, the degree of dryness of the year that controls is a matter of chance. The longer the record period, the greater is the chance that it will include a very dry year. Plotting in even this crude way is an improvement, in that it gives some idea of the range of results and the frequency of recurrence of extreme values.

Probability Paper.—The practical difficulty with the plotting on Fig. 1 is the great curvature of the lines showing the required storage. This difficulty is so great as to make the method unsatisfactory in most cases; but it has been removed by using paper ruled with lines spaced in accordance with a probability curve, or, as it is otherwise called, the normal law of error. The spacing of the lines for this paper was computed from figures taken from probability curve tables, and arranged so that the line which represents the summation of the probability curve, when plotted on it, is straight. If the data for any series correspond strictly with the normal law of error, the points plotted on this paper will all be in a straight line. If the data approximate the normal law of error, the line through the points will approximate a straight line. Even though the deviation from the normal law of error is considerable, a line with only a moderate curvature may represent it fairly well.

In plotting observations on this paper, either of two methods may be used. First, all the results may be divided into classes between certain limits, and the corresponding limits in the other direction may be determined and plotted. This is the better method where the number of terms in the series is large. It cannot be well used with the relatively small number of terms ordinarily constituting a series of storage data. In the second method the whole space is divided into as many vertical strips as there are terms, and the figure for each term, arranged in its order of magnitude, is plotted at the percentage which corresponds to the middle of its strip. That is to say, if there are 50 terms in the series, each is taken to represent 2% of the whole space. The first term will be plotted at the middle of the first 2% strip; that is, on the 1% line; the second term will be plotted in the middle of the second strip, or on the 3% line, etc.

The position for plotting results can be obtained with sufficient accuracy with a 10-in. slide-rule. The decimal position of the mth term in a series of n terms is found to be $P = \frac{2m - 1}{2n}$.

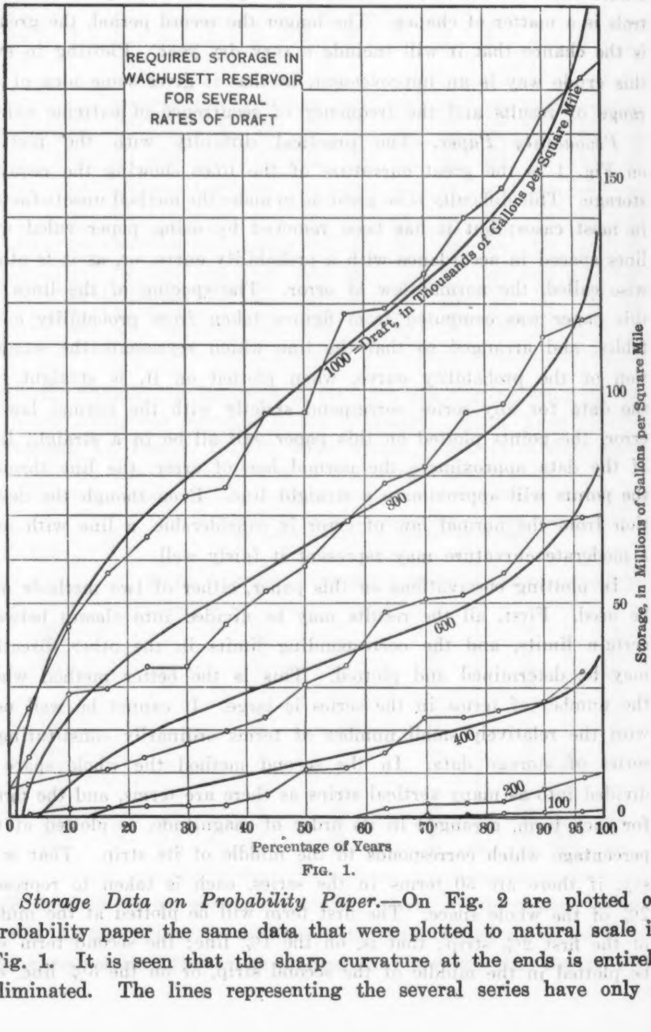

FIG. 1.

Storage Data on Probability Paper.—On Fig. 2 are plotted on probability paper the same data that were plotted to natural scale in Fig. 1. It is seen that the sharp curvature at the ends is entirely eliminated. The lines representing the several series have only a

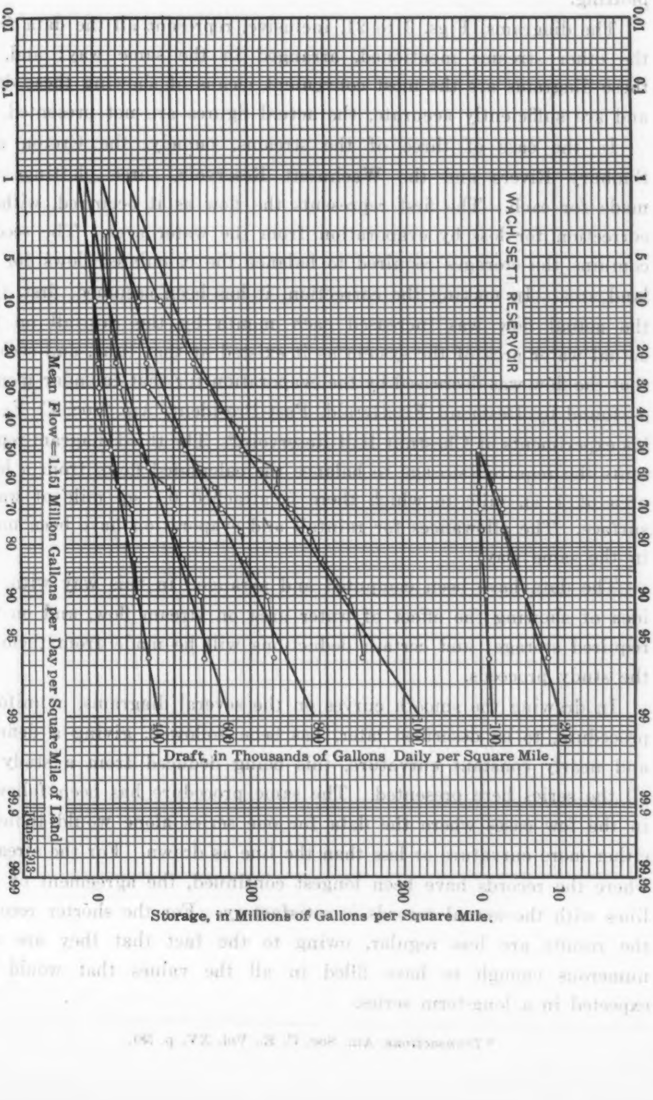
WACHUSETT RESERVOIR
Mean Flow=1.151 Million Gallons per Day per Square Mile of Land
Draft, in Thousands of Gallons Daily per Square Mile.
Storage, in Millions of Gallons per Square Mile.
June 1913

FIG. 2.

moderate curvature, and this is much the same in the different cases. This method represents the data more satisfactorily than arithmetical plotting.

The diagrams, Figs. 3 to 21, inclusive, represent all the data from the other streams mentioned, arranged in the same way; and, as these diagrams are the most convenient means of showing these data, and are sufficiently accurate, the actual figures are not presented.

In the case of three of the streams, namely, the Croton and Sudbury Rivers and the Wachusett Reservoir, three plottings are made for each. The first represents the flow as it occurred, without correction for loss by evaporation from the water area. The second contains the storages required to balance the calculated flows for all land area. In making the correction, it has been assumed, first, that the actual flow was increased each month by the rainfall on the actual water area of the system as it existed at that time, and second, that the flow was decreased by the evaporation for that calendar month, as found by Desmond FitzGerald, Past-President, Am. Soc. C. E., in his experiments at Chestnut Hill Reservoir.* The third diagram represents the required storage to balance the calculated flows from a land area of 1 sq. mile to which there is attached 0.1 sq. mile of water surface. The allowances for rainfall and evaporation have been made in the same way.

The data have been computed and arranged in this way with the idea of showing the effect of water area on stream flow, and on the required storage; and certain deductions will be made from them as the study proceeds.

In drawing the smooth curves on the several diagrams, a uniform procedure, to be described later, has been followed, giving a definite and nearly constant curvature, this being deduced from a study of all the series here presented. The same procedure has been followed in the few cases where the data for one series alone would indicate either more curvature or less than the line as drawn. For the streams where the records have been longest continued, the agreement of the lines with the actual records is satisfactory. For the shorter records the results are less regular, owing to the fact that they are not numerous enough to have filled in all the values that would be expected in a long-term series.

* *Transactions*, Am. Soc. C. E., Vol. XV, p. 581.

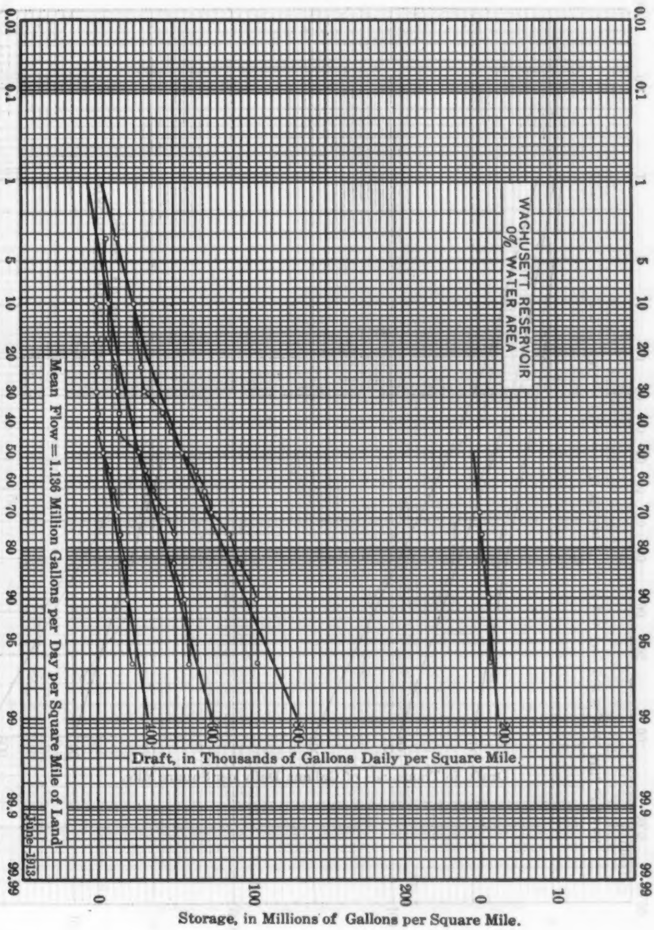
WACHUSETT RESERVOIR
0% WATER AREA
Mean Flow = 1.136 Million Gallons per Day per Square Mile of Land
Draft, in Thousands of Gallons Daily per Square Mile
June 1913
0.01
0.1
1
5
10
20
30
40
50
60
70
80
90
95
99
99.9
99.99
0
100
200
0
10
Storage, in Millions of Gallons per Square Mile.

FIG. 8.

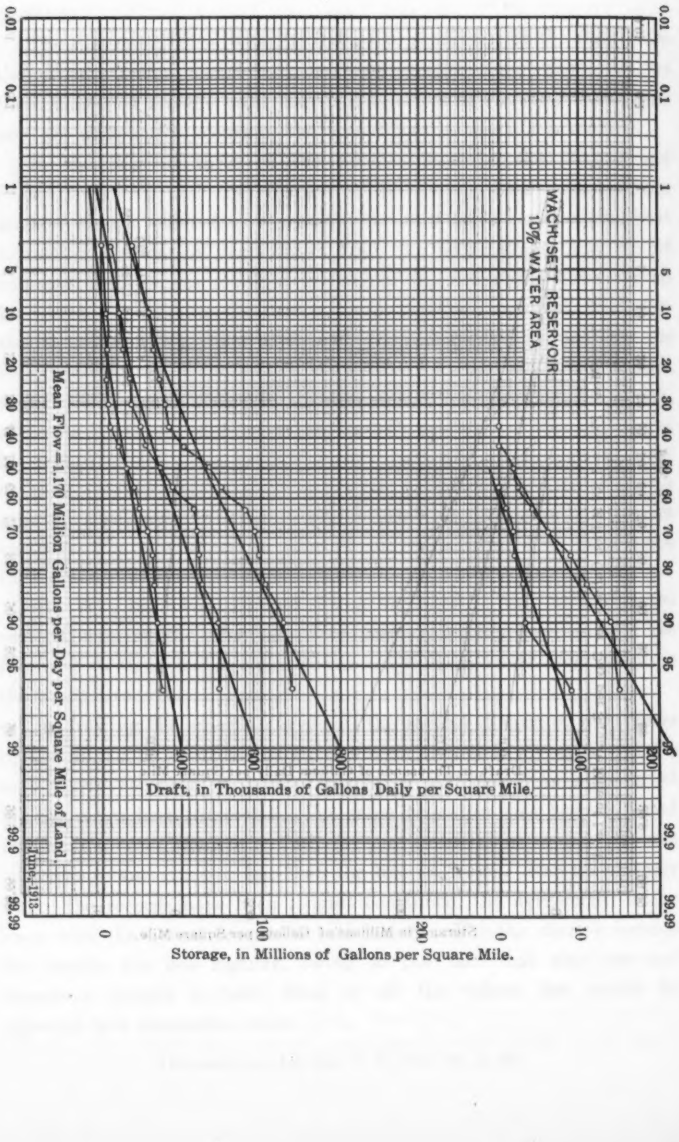
WACHUSETT RESERVOIR
10% WATER AREA
Mean Flow = 1.170 Million Gallons per Day per Square Mile of Land.
June, 1913
Draft, in Thousands of Gallons Daily per Square Mile.
400
600
800
100
200
Storage, in Millions of Gallons per Square Mile.
0
100
200
0
10
0.01
0.1
1
5
10
20
30
40
50
60
70
80
90
95
99
99.9
99.99

FIG. 4.

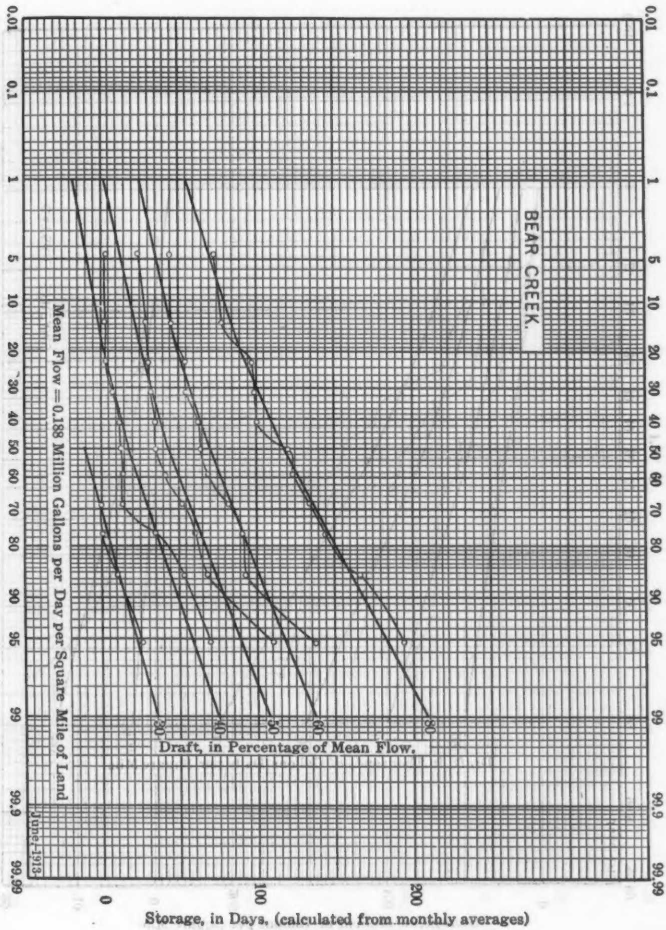
BEAR CREEK.
Mean Flow = 0.188 Million Gallons per Day per Square Mile of Land
Draft, in Percentage of Mean Flow.
30
40
50
60
80
0.01 0.1 1 5 10 20 30 40 50 60 70 80 90 95 99 99.9 99.99
0
100
200
Storage, in Days, (calculated from monthly averages)
June-1913

FIG. 5.

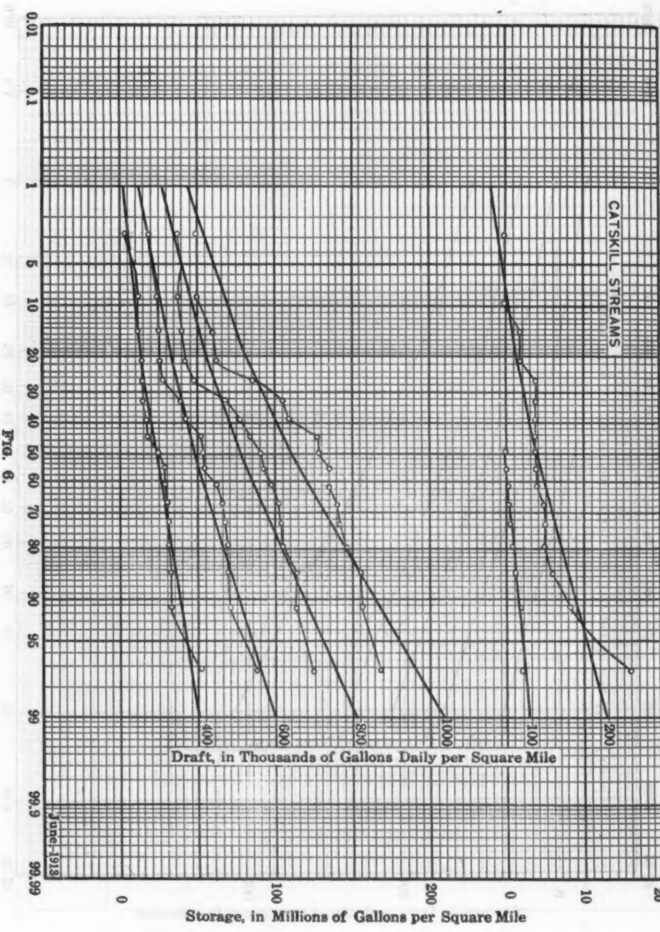
CATSKILL STREAMS
Draft, in Thousands of Gallons Daily per Square Mile
400
600
800
1000
100
200
June-1918
Storage, in Millions of Gallons per Square Mile

Fig. 6.

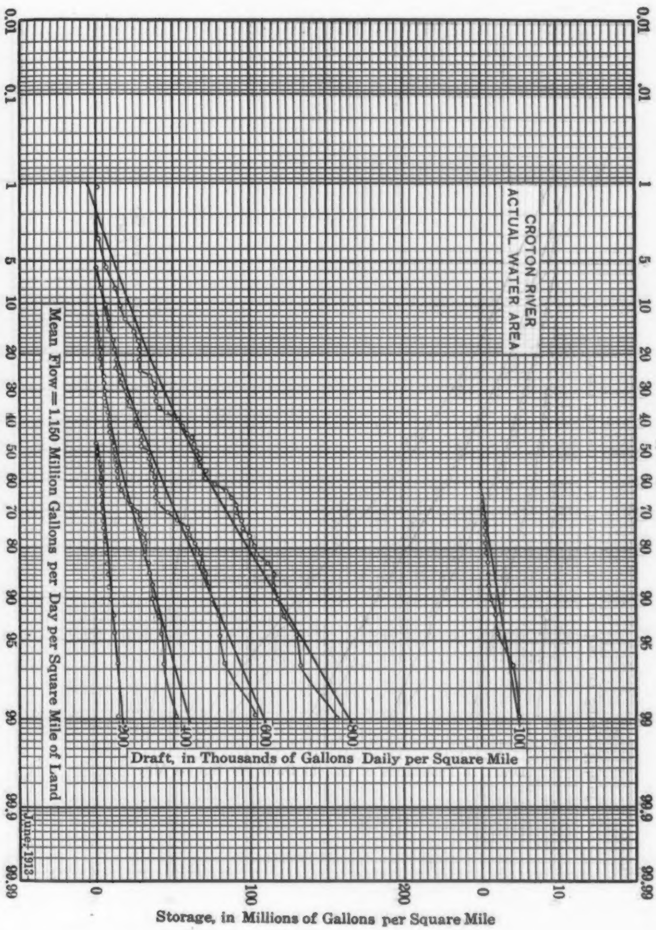
CROTON RIVER
ACTUAL WATER AREA
Mean Flow = 1.150 Million Gallons per Day per Square Mile of Land
Draft, in Thousands of Gallons Daily per Square Mile
200
400
600
800
100
June-1913
Storage, in Millions of Gallons per Square Mile

Fig. 7.

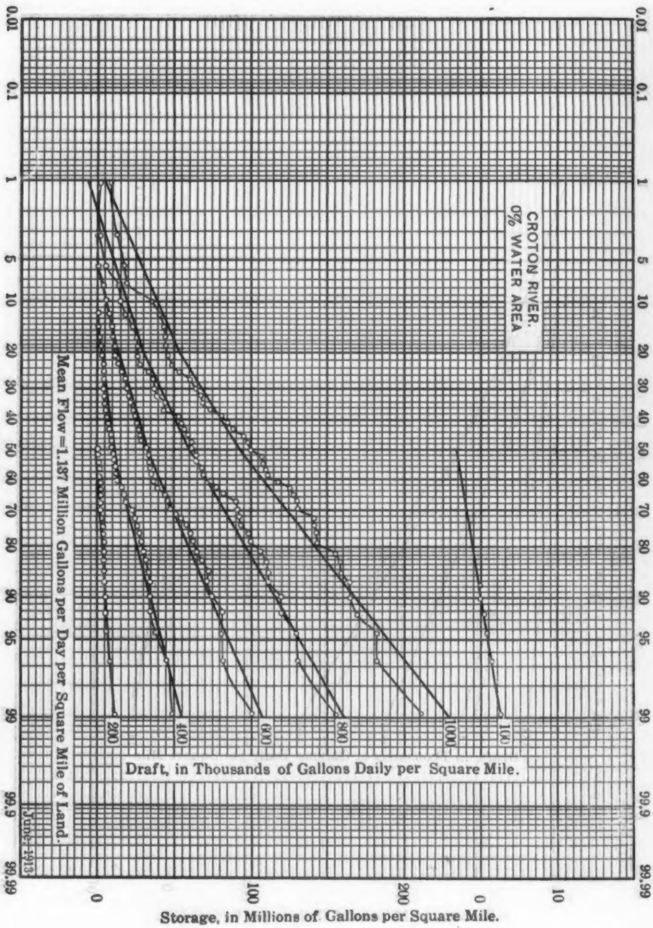
CROTON RIVER.
0% WATER AREA
Mean Flow=1.137 Million Gallons per Day per Square Mile of Land.
Draft, in Thousands of Gallons Daily per Square Mile.
200
400
600
800
1000
100
June 1913
Storage, in Millions of Gallons per Square Mile.

Fig. 8.

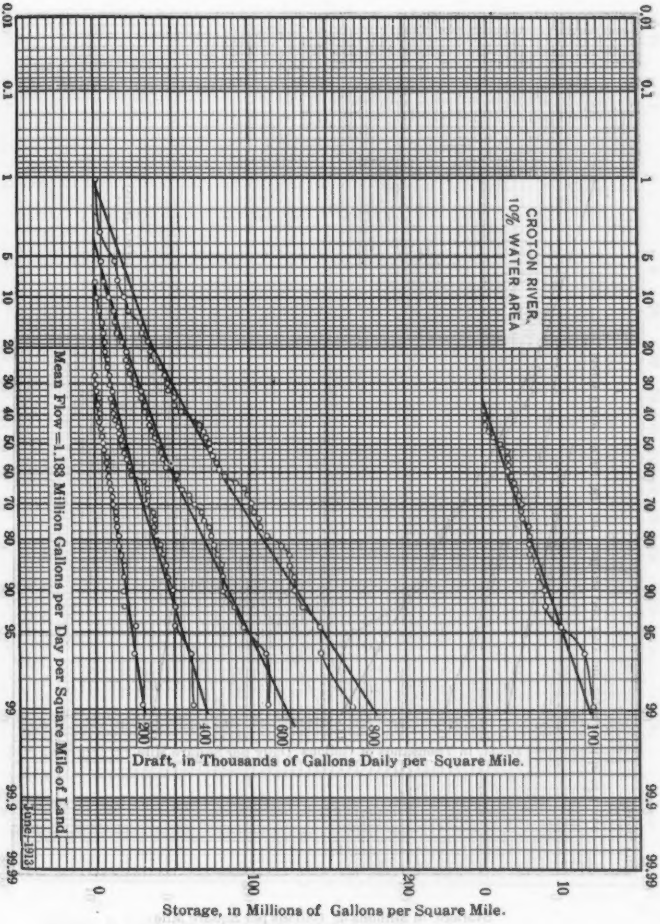
CROTON RIVER.
10% WATER AREA
Mean Flow=1.183 Million Gallons per Day per Square Mile of Land
June-1913
Draft, in Thousands of Gallons Daily per Square Mile.
200
400
600
800
100
0.01
0.1
1
5
10
20
30
40
50
60
70
80
90
95
99
99.9
99.99
0
100
200
0
10
Storage, in Millions of Gallons per Square Mile.

Fig. 9.

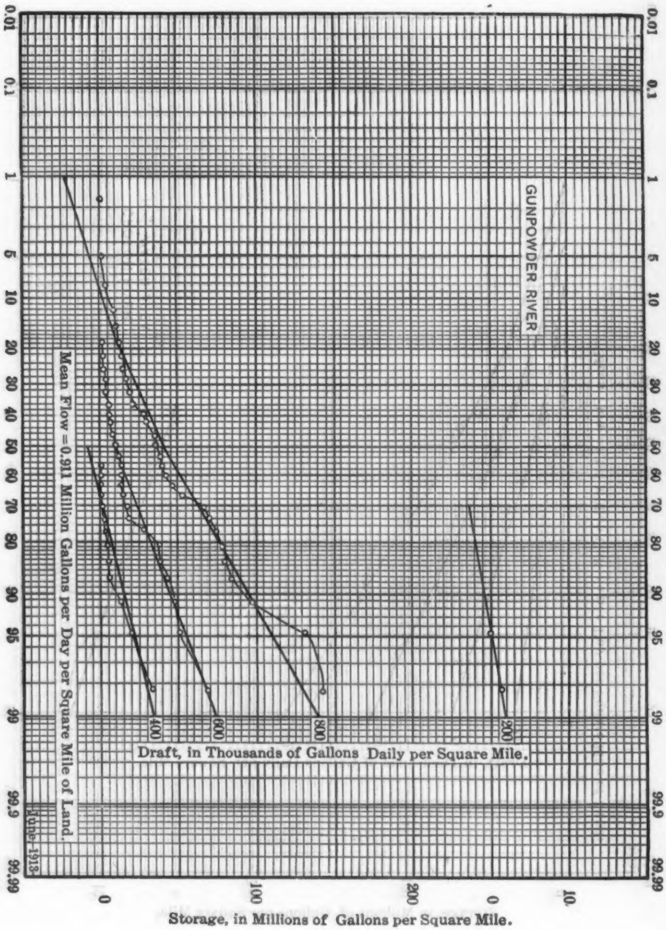
GUNPOWDER RIVER
Mean Flow = 0.911 Million Gallons per Day per Square Mile of Land.
Draft, in Thousands of Gallons Daily per Square Mile.
400
600
800
200
June 1913
0.01 0.1 1 5 10 20 30 40 50 60 70 80 90 95 99 99.9 99.99
0 100 200 0 10.
Storage, in Millions of Gallons per Square Mile.

FIG. 10.

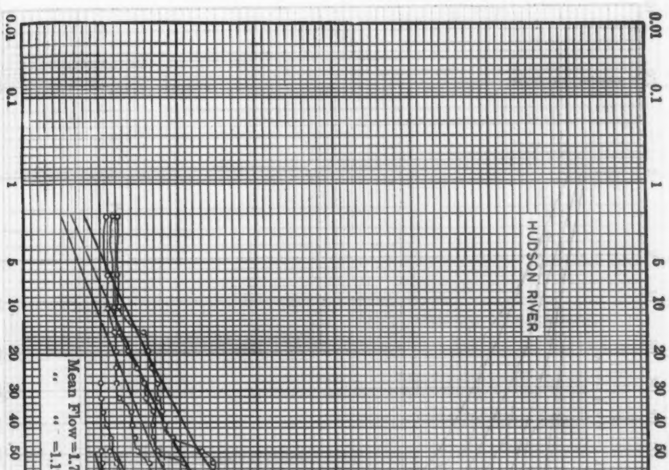
HUDSON RIVER
Mean Flow =1.757 Cubic Feet per Second per Square Mile.
" " =1.171 Million Gallons per Day per Square Mile
Draft, in Percentage of Mean Flow.
28.5
40.0
57.0
68.4
79.8
June 1913
0.01
0.1
1
5
10
20
30
40
50
60
70
80
90
95
99
99.9
99.99
0
100
200
Storage, in Days, (calculated from monthly average).

FIG. 11.

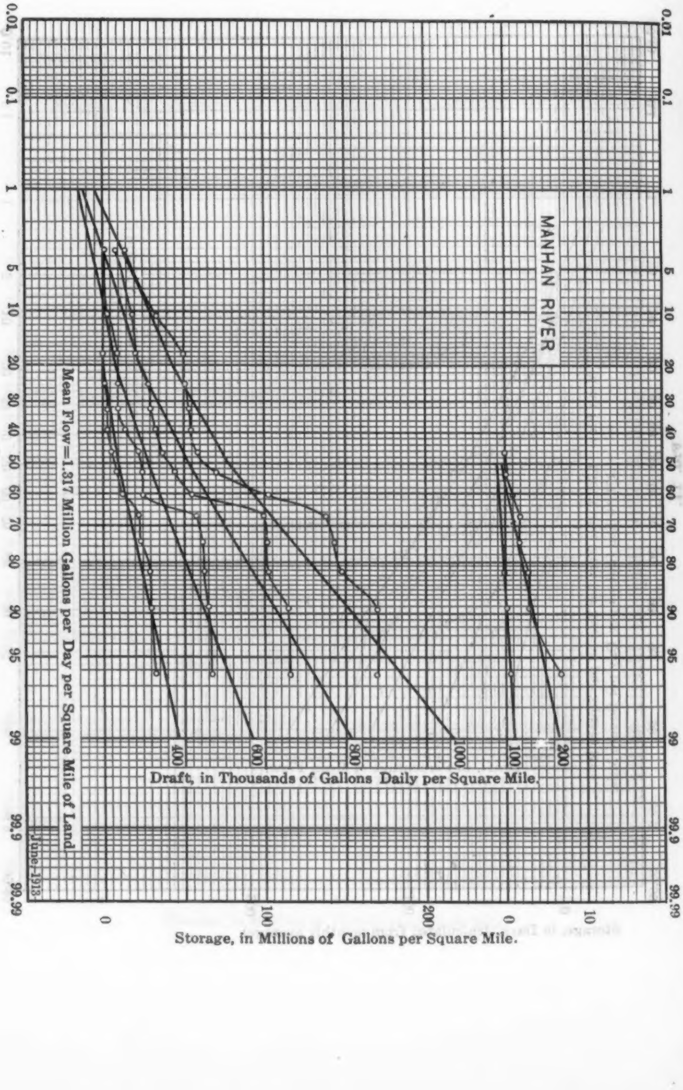
MANHAN RIVER
Mean Flow=1.317 Million Gallons per Day per Square Mile of Land
Draft, in Thousands of Gallons Daily per Square Mile.
400
600
800
1000
100
200
June 1913
0.01
0.1
1
5
10
20
30
40
50
60
70
80
90
95
99
99.9
99.99
0
100
200
0
10
Storage, in Millions of Gallons per Square Mile.

FIG. 12.

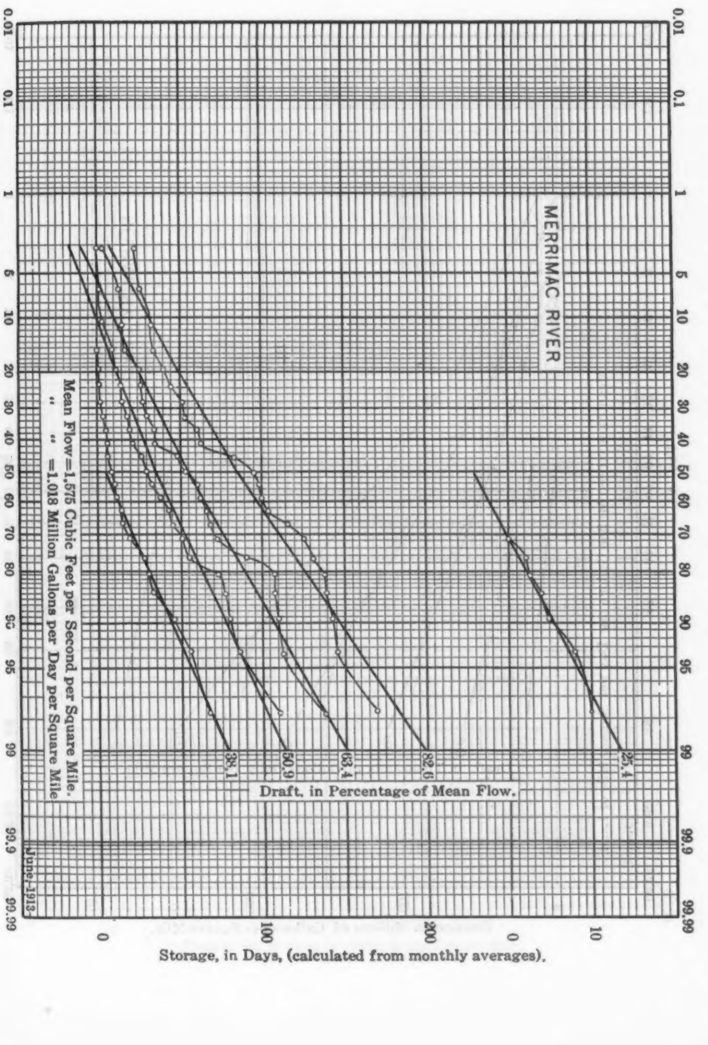
MERRIMAC RIVER
Mean Flow=1.575 Cubic Feet per Second per Square Mile.
" " =1.018 Million Gallons per Day per Square Mile
Draft, in Percentage of Mean Flow.
38.1
50.9
63.4
82.6
25.4
June-1913
0.01 0.1 1 5 10 20 30 40 50 60 70 80 90 95 99 99.9 99.99
0 100 200 0 10
Storage, in Days, (calculated from monthly averages).

Fig. 13.

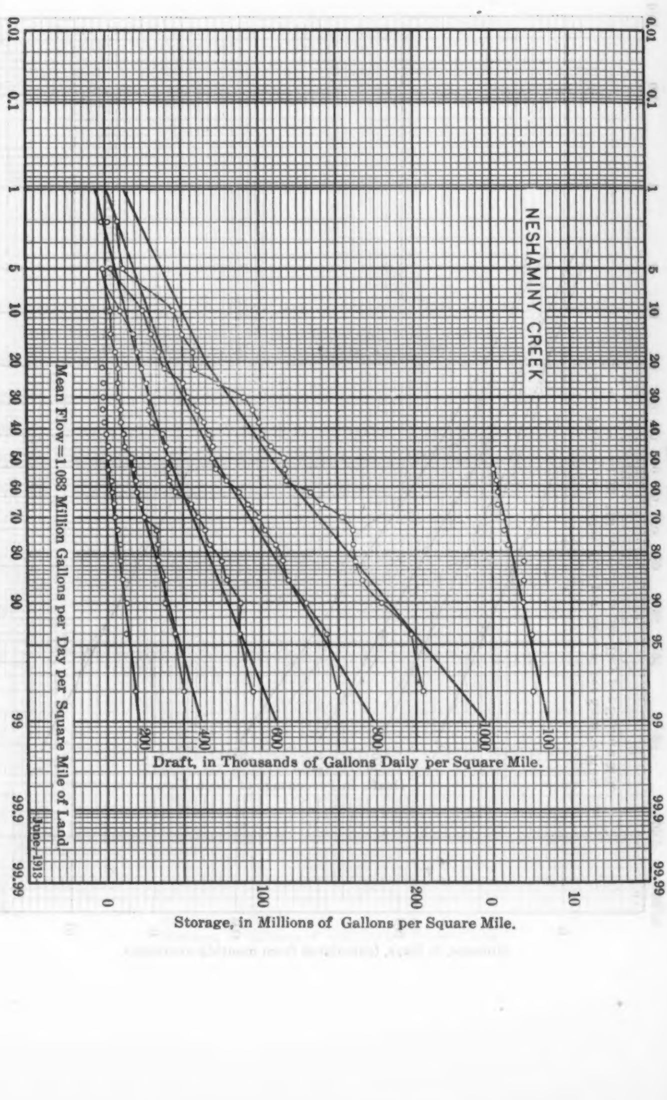
NESHAMINY CREEK
Mean Flow=1.083 Million Gallons per Day per Square Mile of Land
June 1913
Draft, in Thousands of Gallons Daily per Square Mile.
200
400
600
800
1000
100
0.01 0.1 1 5 10 20 30 40 50 60 70 80 90 95 99 99.9 99.99
0 100 200 0 10
Storage, in Millions of Gallons per Square Mile.

FIG. 14.

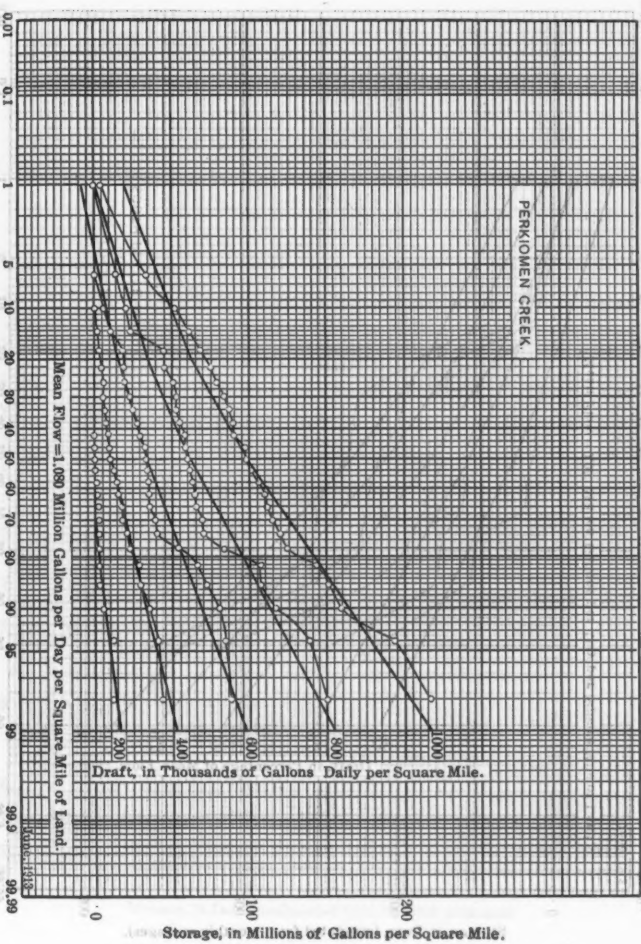
PERKIOMEN CREEK.
Mean Flow=1.080 Million Gallons per Day per Square Mile of Land.
Draft, in Thousands of Gallons Daily per Square Mile.
200
400
600
800
1000
June 1913
0.01
0.1
1
5
10
20
30
40
50
60
70
80
90
95
99
99.9
99.99
0
100
200
Storage, in Millions of Gallons per Square Mile.

FIG. 15.

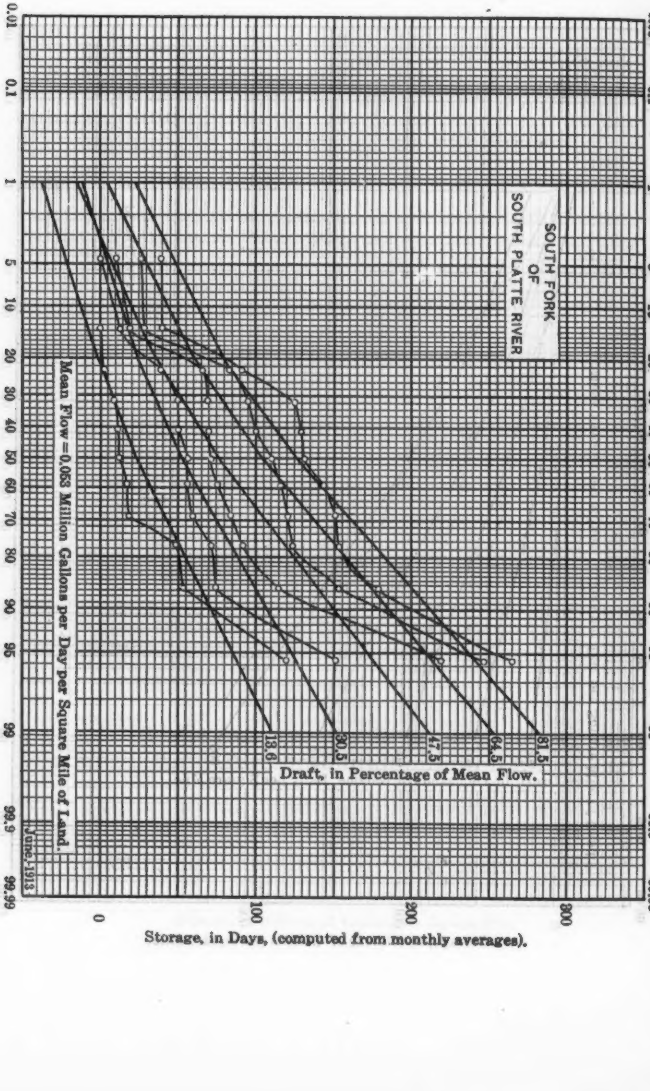
SOUTH FORK
OF
SOUTH PLATTE RIVER
Mean Flow = 0.053 Million Gallons per Day per Square Mile of Land.
Draft, in Percentage of Mean Flow.
13.6
30.5
47.5
64.5
81.5
June, 1913
0.01
0.1
1
5
10
20
30
40
50
60
70
80
90
95
99
99.9
99.99
0
100
200
300
Storage, in Days, (computed from monthly averages).

FIG. 16.

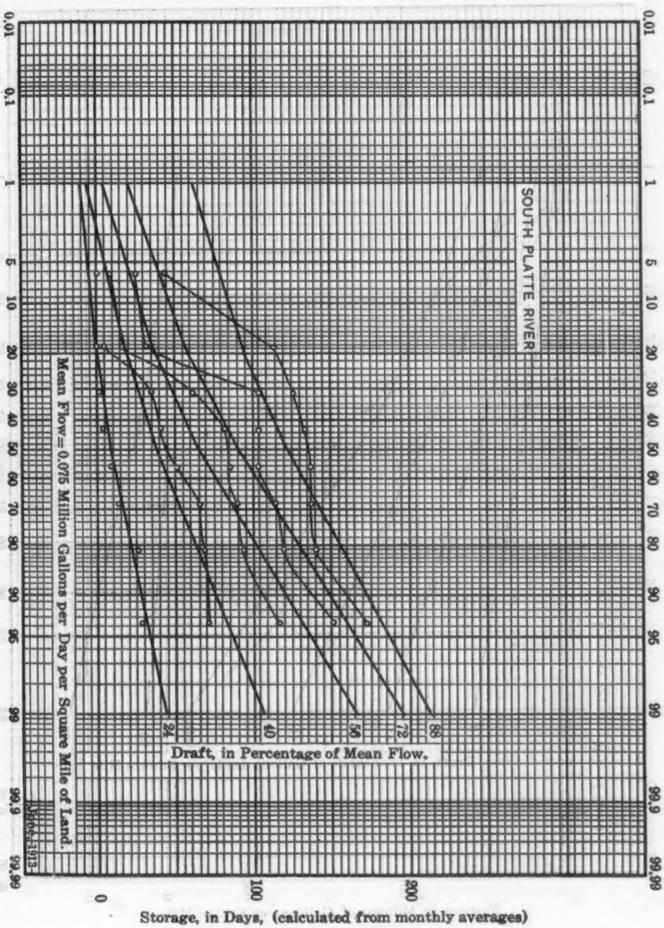
SOUTH PLATTE RIVER
Mean Flow=0.075 Million Gallons per Day per Square Mile of Land.
Draft, in Percentage of Mean Flow.
24
40
56
72
88
0.01
0.1
1
5
10
20
30
40
50
60
70
80
90
95
99
99.9
99.99
0
100
200
Storage, in Days, (calculated from monthly averages)

FIG. 17.

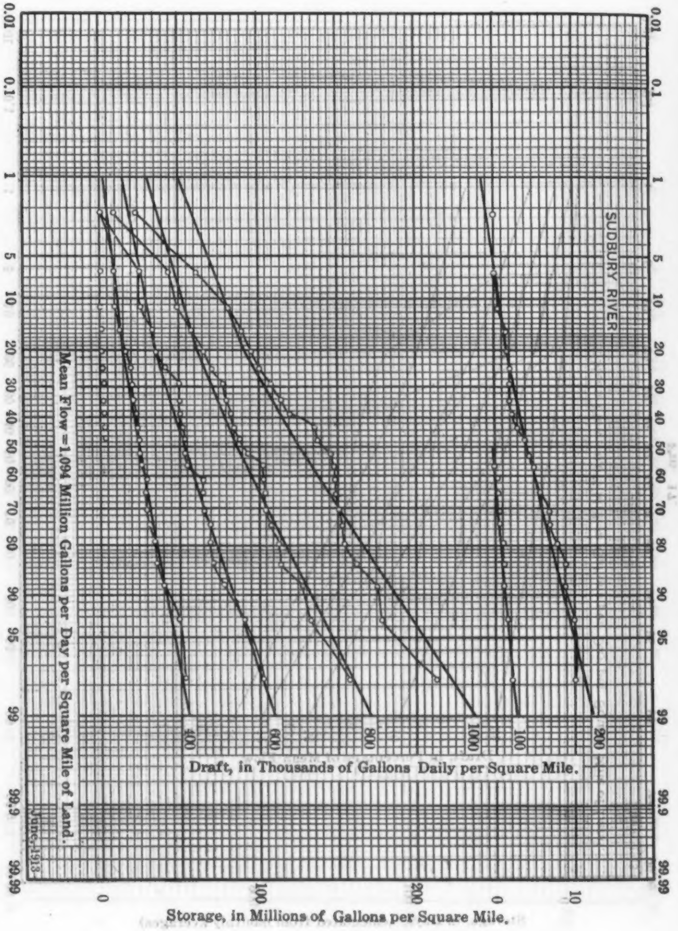
SUDBURY RIVER
Mean Flow = 1.094 Million Gallons per Day per Square Mile of Land.
Draft, in Thousands of Gallons Daily per Square Mile.
400
600
800
1000
100
200
June, 1913
0.01
0.1
1
5
10
20
30
40
50
60
70
80
90
95
99
99.9
99.99
0
100
200
0
10
Storage, in Millions of Gallons per Square Mile.

Fig. 18.

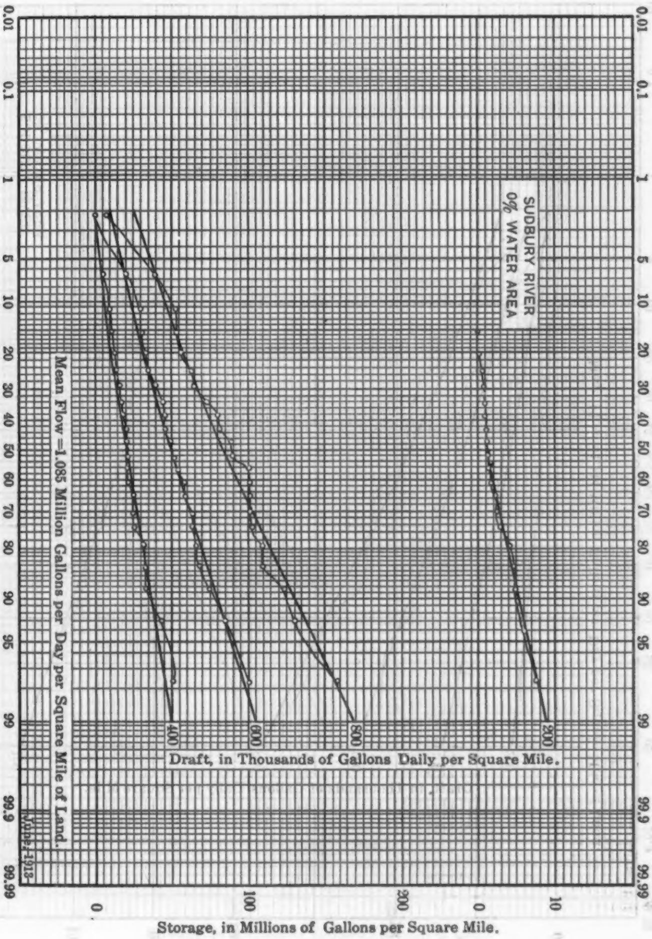
SUDBURY RIVER
0% WATER AREA
Mean Flow =1.085 Million Gallons per Day per Square Mile of Land.
Draft, in Thousands of Gallons Daily per Square Mile.
200
800
600
400
June, 1913
0.01 0.1 1 5 10 20 30 40 50 60 70 80 90 95 99 99.9 99.99
0 100 200 0 10
Storage, in Millions of Gallons per Square Mile.

FIG. 19.

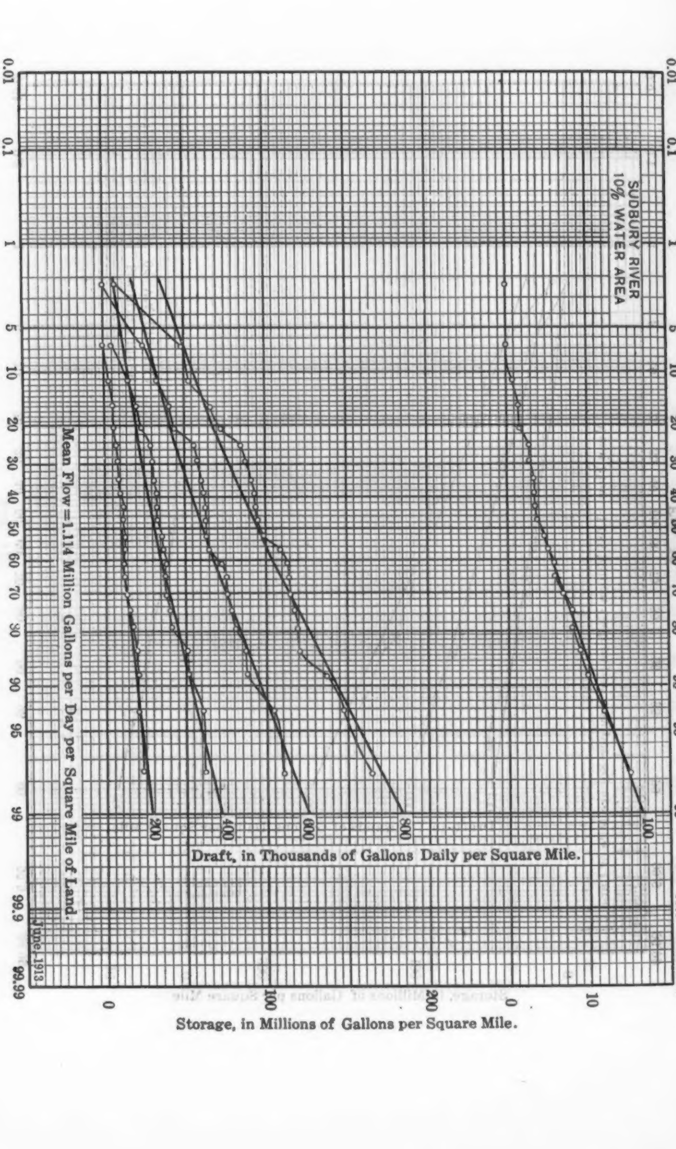
SUDBURY RIVER
10% WATER AREA
Mean Flow = 1.114 Million Gallons per Day per Square Mile of Land.
Draft, in Thousands of Gallons Daily per Square Mile.
200
400
600
800
100
June 1913
0.01
0.1
1
5
10
20
30
40
50
60
70
80
90
95
99
99.9
99.99
0
100
200
0
10
Storage, in Millions of Gallons per Square Mile.

FIG. 20.

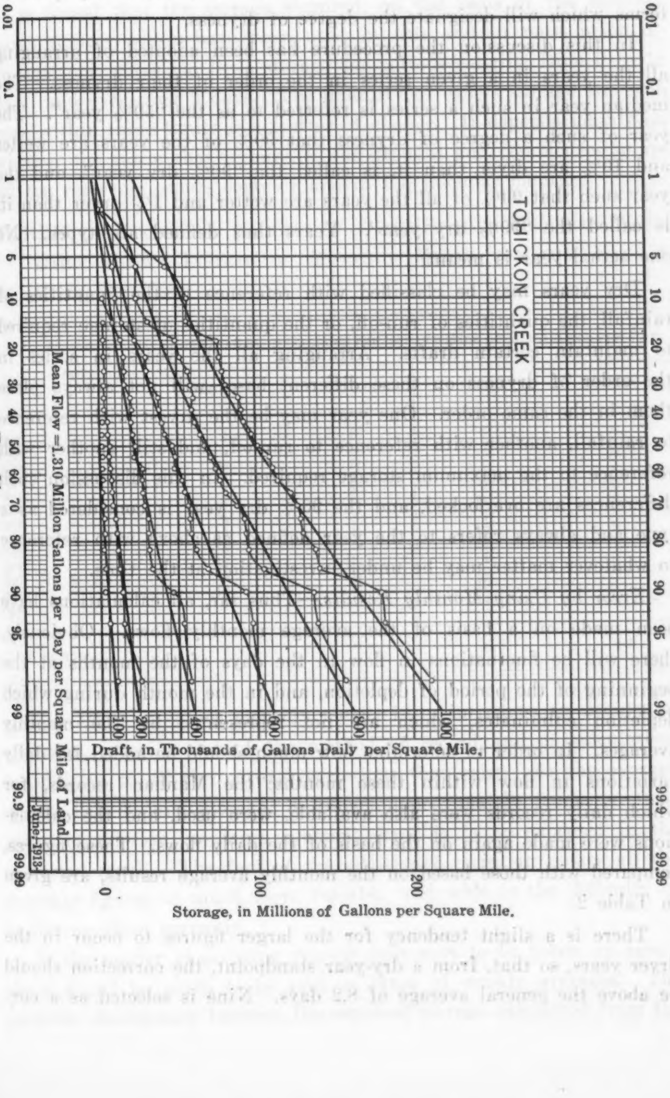
TOHICKON CREEK
Mean Flow = 1.310 Million Gallons per Day per Square Mile of Land
Draft, in Thousands of Gallons Daily per Square Mile.
100
200
400
600
800
1000
June-1913
0.01
0.1
1
5
10
20
30
40
50
60
70
80
90
95
99
99.9
99.99
0
100
200
Storage, in Millions of Gallons per Square Mile.

FIG. 21.

Definition of a Dry Year.—Dry years recur at intervals. The dryer the year the longer is the probable interval of its recurrence. For intelligent discussion, it is necessary to define a dry year in terms which will designate the degree of dryness.

In this discussion the procedure has been adopted of arranging all the years in a given series in the order of their dryness. The median year in such a series is referred to as the "50% year". The year of such a degree of dryness that 90% of the years are wetter and 10% are dryer than it, is called the "90% dry year", and the year such that 99% of all the years are wetter and 1% dryer than it, is called the "99% dry year". Years thus defined are types. No one actual year is meant.

Dry years may be classified with reference to the quantities of rainfall, the quantities of run-off, or the quantities of storage required to maintain certain drafts. Arranging all the years in series in the order of dryness on these different bases will not always place them in the same order. One year may be the dryest with reference to rainfall, another with reference to run-off, and still another with reference to the maximum storage required. In this discussion such differences are overlooked, and the 90% dry year is considered as a type and always refers to the year defined as above with reference to whatever matter may be under investigation at the time.

Error by Using Monthly Results.—Thus far, all calculations have been made on a basis of the average monthly flows. Obviously, there will be fluctuations in flow in the days of the months at the beginning of the period of depletion, and in the month during which depletion culminates, which are not represented by the monthly averages. In order to determine how much should be added for daily variations in flow within these months, the Manhan records, for which daily records were also available, were used, and the calculations were made again on the basis of the daily flows. These figures, compared with those based on the monthly average results, are given in Table 2.

There is a slight tendency for the larger figures to occur in the dryer years, so that, from a dry-year standpoint, the correction should be above the general average of 8.2 days. Nine is selected as a cor-

rection to be added to all monthly figures. This will be called the "daily storage".

Effect of Using the Monthly Basis on the Rest of the Calculation.—It is found that the storage required, on the basis of the monthly mean figures which have been used in this and in most prior investigations, gives too little storage for periods ranging from 0 to 18 days, and averaging 9 days. The quantity of excess storage for each year within these limits is a matter of chance. It depends mainly on whether the storm that terminates the drought and commences to refill the reservoir occurs early or late in the calendar month.

TABLE 2.—ADDITIONAL NUMBER OF DAYS' STORAGE REQUIRED WHEN DAILY FLOWS INSTEAD OF MONTHLY MEANS FOR THE MANHAN RIVER ARE USED.

Year.	FOR DRAFTS, IN GALLONS PER SQUARE MILE OF LAND AREA PER DAY.			
	400 000	600 000	800 000	1 000 000
1897	2	8	6	4
1898	5	3	5	6
1899	0	0	7	7
1900	5	7	3	5
1901	5	8	9	12
1902	17	8	11	13
1903	5	8	6	7
1904	10	8	12	9
1905	5	5	15	11
1906	12	12	4	5
1907	5	8	12	18
1908	10	8	10	11
1909	7	17	14	16
Average	6.8	7.7	8.8	9.5

The effect of using monthly means is to introduce an accidental variation, growing out of the method of record and of calculation, in addition to all the natural variations that exist. As the average excess, called daily storage, is known approximately from the foregoing calculation, no large constant error is to be expected in the corrected result. The fact of the additional variation makes all the monthly figures so much more variable, and adds to the difficulty of analyzing them correctly.

If the matter of securing run-off data were to be taken up again, there would be much to be said in favor of weekly averages. The probable discrepancy between the required storage calculated from the

weekly averages and the daily results would be so small that it could be overlooked. The weekly averages would be easier of analysis than the daily results, and would serve all practical purposes. The weeks are also all of the same length, and the slight errors introduced by the fact that the months are not of the same length would be eliminated.

As nearly all the data now available are on a monthly basis, and as the daily records could only be secured and analyzed with great labor (and, in some instances, not at all), this matter is not open for reconsideration at this time. Attention is called to it with the view of raising the question whether in future it would not be better to use the weekly instead of the monthly basis. The weekly basis has been used always for the record of the flow of the Merrimac River, kept by the Essex Company at Lawrence, for the Connecticut River at Holyoke, and the Pequannock River by the City of Newark.

The importance of making the correction for daily results, when monthly records are used for the basis of calculation, will be realized when it is stated that, for most of the Eastern streams investigated, this correction amounts to more than the allowance for evaporation.

Method of Least Squares.—Some of the methods of least squares have been found to be applicable to the data of flows and storage. Without explaining the methods found in textbooks, the following fundamental definitions may be given:

The "mean" of a series of terms is the arithmetical average of all the terms. The "median" is the middle term of the series. The "variation" of any term is the difference between that term and the mean. The "standard variation" is the square root of the mean square of the variations of all the terms. The "coefficient of variation" is the ratio of the standard variation to the mean. The "average variation" is the arithmetical average of all the variations. The "probable error" is that variation which is exceeded by one-half the variations, or it is the median of the variations.

The standard variation and the coefficient of variation have generally been used as a basis for calculation. As the tables of the curve of normal error are most commonly given in terms of the probable error, it is often necessary to change one to the other. With data following the law of normal error, the ratios between the standard variation, the average variation, and the probable error are constant, and are as follows:

Standard variation.....	1.000	1.253	1.483
Average variation......	0.798	1.000	1.183
Probable error.........	0.6745	0.8453	1.000

Data to Which This Method is Not Applicable.—This method of analysis cannot be applied safely to any series of results in which there is more than one zero. There are many important series of this character to be investigated. Such series include the storages required to maintain low rates of draft where, in wet years, the natural flow does not fall below the assumed rate of draft, and when, therefore, no storage is required. It also cannot be applied unless all the terms of the series are available. For instance, good data may be at hand as to the flow in a certain number of dry years, but without corresponding data for intervening years of average, or more than average, flow. For both these cases other methods of handling the data must be found.

In cases of series where there are several zeros, it is possible to get a fair approximation to the standard variation by dividing the difference between the storage for the 99% year and the 50% year, as determined graphically from the plotting, by 2.6,* and to approximate the mean storage by adding 0.1 of the standard variation thus found to the storage for the 50% year. Figures thus obtained are starred in the tables which follow to distinguish them from those found by the ordinary arithmetical procedure. By this approximate method, reliable values are obtained which are needed to round out the study and would not otherwise be available.

The adjusted mean storage computed in this way, of course, is not a true mean; but it bears the relation to that part of the series which is available that the mean of the whole series would bear if the whole series were available. The reason the mean obtained by averaging the actual figures does not bear this relation is that, if the whole series were available, some of the terms would be represented by negative numbers. These negative numbers cannot be computed and are not known, but the fact that they would exist in a theoretically complete series introduces a disturbing element if the actual mean is used.

In Table 3 is presented a concise summary of the storage data taken from Figs. 2 to 21 by the foregoing methods.

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* If the normal law of error applied exactly, this factor would be 2.33. The value here given is deduced from actual results obtained from data considered and shown in Fig. 29.

TABLE 3.—Monthly Storage Data for Several Streams. (Each Year by Itself. No Cumulative Storage Included.)

(1)	(2)	(3)	(4)	(5)	(6)
	Assumed Rate of Draft.		Storage Required, in Days' Flow at Assumed Rate of Draft, with Nine Days Added to Monthly Figures to Cover Daily Fluctuations.		
Stream.	Gallons per day per square mile of land area.	As portion of mean flow.	Mean.	In 95% year.	Standard variation in days' storage.
Wachusett, actual	200 000	0.174	11*	46	20.2*
	400 000	0.348	40	89	27.0
	600 000	0.521	66	126	34.8
	800 000	0.695	88	157	39.0
	1 000 000	0.870	110	187	43.0
Wachusett, no water area	200 000	0.176	7*	17	5.0*
	400 000	0.352	24*	71	28.1*
	600 000	0.528	58	112	30.7
	800 000	0.704	82	145	36.4
Wachusett, 0.1 sq. mile water area	200 000	0.171	23*	88	38.5*
	400 000	0.342	53	111	32.8
	600 000	0.513	75	141	38.5
	800 000	0.684	95	165	40.8
Bear Creek	56 000	0.300	1*	32	17.7*
	75 000	0.400	31	68	21.2
	94 000	0.500	57	100	24.2
	112 000	0.600	83	127	26.1
	150 000	0.800	131	193	35.4
Catskill streams	100 000	0.071	10*	29	11.5*
	200 000	0.143	31*	60	17.3*
	400 000	0.286	69	117	27.8
	600 000	0.428	94	152	33.4
	800 000	0.572	112	174	36.1
	1 000 000	0.715	125	190	38.1
Croton, actual	100 000	0.087	3*	40	22.4*
	200 000	0.174	17*	69	28.9*
	400 000	0.348	44*	124	46.2*
	600 000	0.522	72*	154	47.5*
	800 000	0.695	93	178	49.0
Croton, no water area	200 000	0.176	7*	48	25.0*
	400 000	0.352	38*	115	44.2*
	600 000	0.527	71*	154	48.0*
	800 000	0.703	91	175	49.0
	1 000 000	0.870	108	200	52.0
Croton, 0.1 sq. mile water area	100 000	0.085	33*	110	25.7*
	200 000	0.169	44*	130	50.0*
	400 000	0.338	64*	150	50.0*
	600 000	0.507	82*	174	52.6*
	800 000	0.676	102	193	52.2
Gunpowder	200 000	0.220	—10*	9	11.6*
	400 000	0.439	— 5*	62	40.3*
	600 000	0.658	30*	100	41.0*
	800 000	0.877	65	147	46.5
Hudson †	324 000	0.285	7*	36	17.0*
	454 000	0.400	22*	71	28.5*
	648 000	0.570	47	106	34.4
	775 000	0.684	63	129	38.6
	905 000	0.796	77	152	43.2

*** Adjusted values which have been inferred from the line drawn to represent the data.**
† Includes water area.

TABLE 3.—(*Continued.*)

(1) Stream.	(2) Assumed Rate of Draft. Gallons per day per square mile of land area.	(3) Assumed Rate of Draft. As portion of mean flow.	(4) Storage Required, in Days' Flow at Assumed Rate of Draft, with Nine Days Added to Monthly Figures to Cover Daily Fluctuations. Mean.	(5) Storage Required, in Days' Flow at Assumed Rate of Draft, with Nine Days Added to Monthly Figures to Cover Daily Fluctuations. In 95% year.	(6) Standard variation in days' storage.
Manhan	200 000	0.152	10*	33	13.1*
	400 000	0.304	38*	99	35.6*
	600 000	0.455	61	132	41.5
	800 000	0.607	82	165	48.1
	1 000 000	0.760	99	186	51.8
Merrimac †	259 000	0.254	6*	17	6.7*
	388 000	0.381	16*	66	29.3*
	519 000	0.509	47	100	30.2
	646 000	0.634	67	132	37.2
	842 000	0.826	97	175	44.3
Neshaminy	100 000	0.092	12*	59	27.0*
	200 000	0.184	32*	90	34.6*
	400 000	0.369	56*	130	43.1
	600 000	0.554	84	160	44.0
	800 000	0.738	104	190	48.8
	1 000 000	0.924	122	214	53.0
Perkiomen	200 000	0.185	—2*	59	36.5*
	400 000	0.370	43*	109	38.5*
	600 000	0.555	70	141	40.2
	800 000	0.740	91	169	44.1
	1 000 000	0.926	112	192	45.5
South Fork of South Platte	7 200	0.136	36*	94	33.2*
	16 200	0.305	66	133	37.2
	25 000	0.475	90	181	52.5
	34 000	0,645	119	219	57.4
	43 000	0.815	141	246	60.3
South Platte	18 000	0.240	18*	43	13.8*
	30 000	0.400	45	96	26.1
	42 000	0.560	70	146	35.7
	54 000	0.720	92	175	40.8
Sudbury, actual	100 000	0.091	9*	28	10.8*
	200 000	0.183	31*	59	16.0*
	400 000	0.366	73	125	30.7
	600 000	0.548	101	164	37.3
	800 000	0.732	122	192	40.4
	1 000 000	0.915	139	215	43.2
Sudbury, no water area	200 000	0.184	16*	42	15.0*
	400 000	0.368	63	114	28.8
	600 000	0.553	94	157	36.0
	800 000	0.736	117	188	40.8
Sudbury, 0.1 sq. mile water area	100 000	0.090	56*	139	46.4*
	200 000	0.180	67*	124	32.7*
	400 000	0.359	95	159	37.3
	600 000	0.539	116	187	41.4
	800 000	0.718	132	205	42.7
Tohickon	100 000	0.076	13*	79	38.5*
	200 000	0.152	42*	99	32.7*
	400 000	0.305	69	130	35.6
	600 000	0.458	87	160	42.3
	800 000	0.610	100	178	44.4
	1 000 000	0.764	115	193	44.7

* Adjusted values which have been inferred from the line drawn to represent the data.
† Includes water area.

The figures in Table 3 have been plotted, forming three diagrams. The assumed rate of draft, as a fraction of the mean flow, as shown in Column 3, is used as a base for each. In Fig. 22 are plotted the figures in Column 4, showing the mean storage required in days' flow for each of the streams. In Fig. 23 the figures in Column 5 are plotted, showing the storage required in a 95% dry year, and Fig. 24 shows the figures in Column 6 for standard variation in days' storage. In plotting these results, the corrected figures for no water surface for the Sudbury, Croton, and Wachusett are used, as being, on the whole, most suitable for this purpose.

It is seen by inspection of Figs. 22 and 23 that the number of days' storage required in different streams varies considerably, but the lines representing the required storage at different rates of draft are rather strikingly parallel with each other. In other words, an increased rate of draft requires nearly the same increase in the quantity of storage on all the streams. This is shown further in Tables 4, 5, and 6.

TABLE 4.—Difference Between the Average Number of Days' Storage Required at Several Rates of Draft, and the Storage Required for 50% of the Mean Flow for Different Streams for the Mean Year.

Stream.	Number of years in record.	Difference Between Required Storage and Storage at 50 Per Cent.					
		0.20 use.	0.30 use.	0.40 use.	0.60 use.	0.70 use.	0.80 use.
Bear Creek	11	....	56	26	26	50	74
Catskill	17	57	31	14	11	21	30
Croton	45	55	38	19	13	25	36
Gunpowder	29	....	....	....	16	32	48
Hudson	23	....	27	14	15	29	41
Manhan	14	48	30	14	14	25	36
Merrimac	23	45	35	25	16	32	48
Neshaminy	25	42	28	15	13	24	34
Perkiomen	25	60	36	15	13	25	36
South Fork	11	46	29	15	17	32	45
South Platte	8	29	32	15	15	29	46
Sudbury	22	66	40	18	14	26	38
Tohickon	25	41	23	11	8	18	27
Wachusett	15	44	34	20	15	28	42
Weighted average	293	51.1	33.5	17.0	14.1	27.2	39.4

Table 7 shows the number of days' storage required to maintain a 50% draft in an average year and in a 95% dry year, scaled from Figs. 22 and 23, and the additional quantity required for the latter over the former.

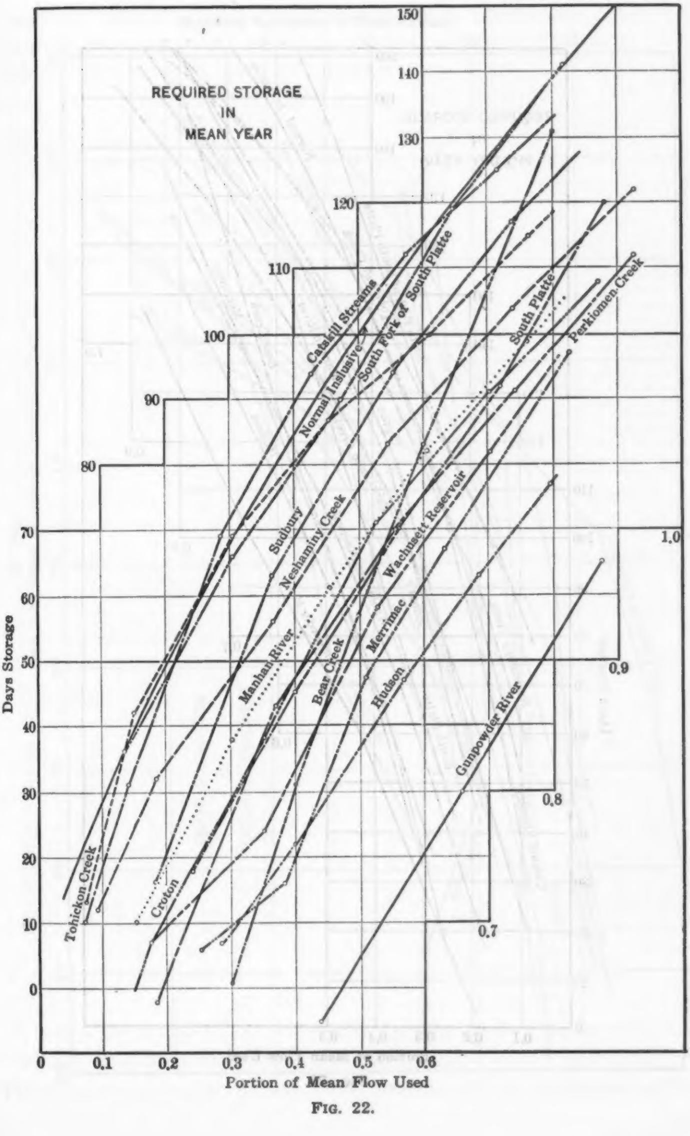

REQUIRED STORAGE
IN
MEAN YEAR
Days Storage
Portion of Mean Flow Used
Catskill Streams
Normal Inclusive
South Fork of South Platte
South Platte
Perkiomen Creek
Sudbury
Neshaminy Creek
Wachusett Reservoir
Manhan River
Bear Creek
Merrimac
Hudson
Gunpowder River
Tohickon Creek
Croton
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

FIG. 22.

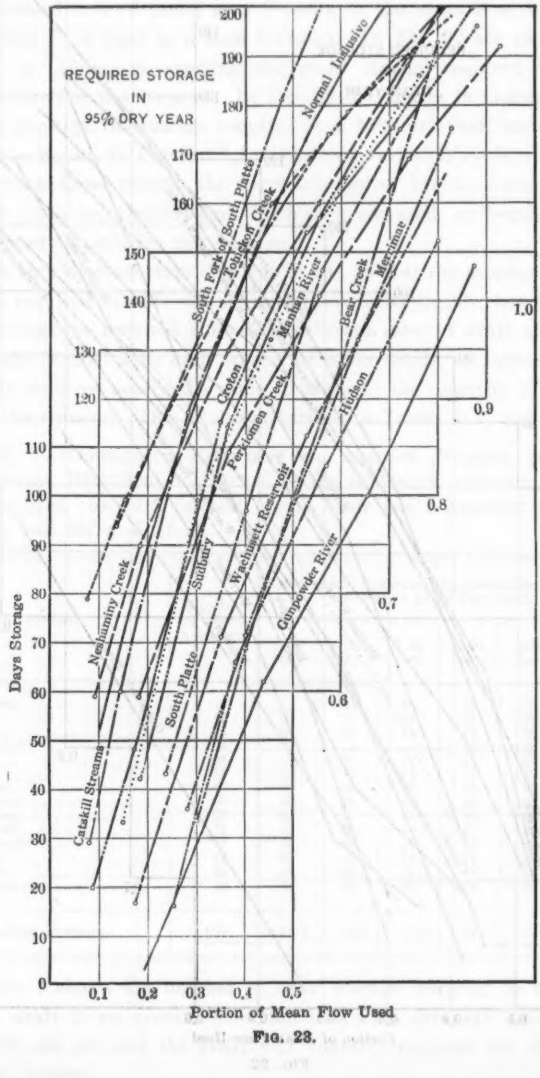

REQUIRED STORAGE
IN
95% DRY YEAR
Days Storage
Portion of Mean Flow Used
Normal Inclusive
South Fork of South Platte
Tohickon Creek
Manhan River
Bear Creek
Merrimac
Croton
Perkiomen Creek
Hudson
Neshaminy Creek
Sudbury
Wachusett Reservoir
South Platte
Gumpowder River
Catskill Streams
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

FIG. 23.

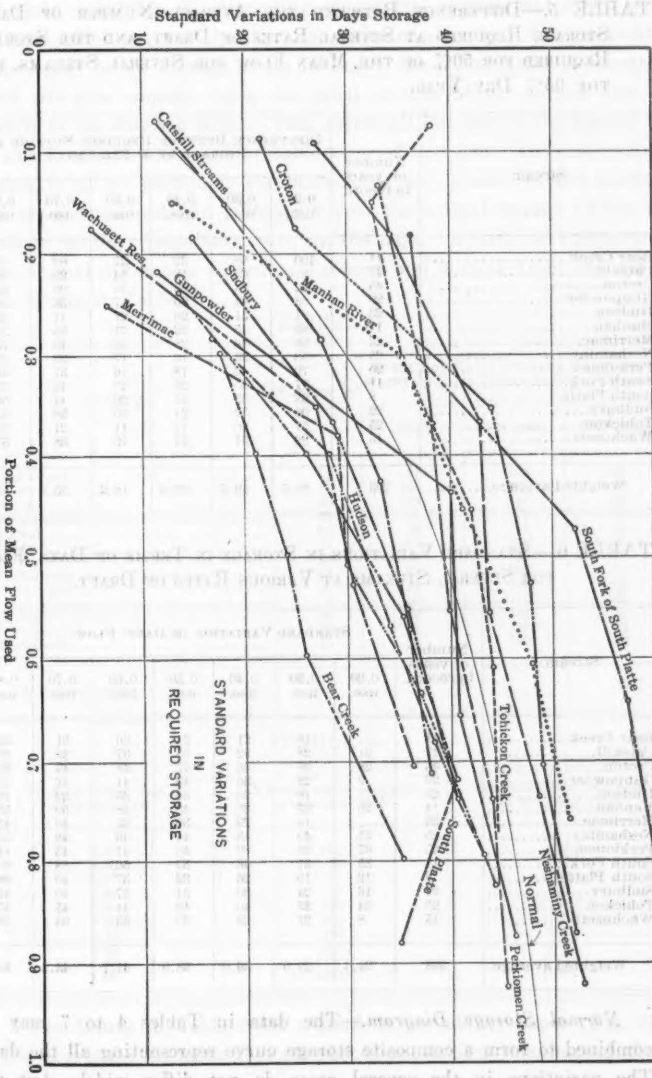

Standard Variations in Days Storage
0
10
20
30
40
50
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
Portion of Mean Flow Used
Catskill Streams
Croton
Wachusett Res.
Sudbury
Manhan River
Gunpowder
Merrimac
Hudson
Bear Creek
South Platte
Tohickon Creek
South Fork of South Platte
Neshaminy Creek
Normal
Perkiomen Creek
STANDARD VARIATIONS
IN
REQUIRED STORAGE

FIG. 24.

TABLE 5.—Difference Between the Average Number of Days' Storage Required at Several Rates of Draft, and the Storage Required for 50% of the Mean Flow for Several Streams, for the 95% Dry Year.

Stream.	Number of years in record.	Difference Between Required Storage and Storage at 50 Per Cent.					
		0.20 use.	0.30 use.	0.40 use.	0.60 use.	0.70 use.	0.80 use.
Bear Creek	11	100	68	32	27	60	93
Catskill	17	81	43	18	14	25	36
Croton	45	92	54	23	14	26	38
Gunpowder	29	68	44	20	17	36	58
Hudson	23	81	51	20	21	41	62
Manhan	14	89	45	22	21	36	50
Merrimac	23	98	64	28	25	49	71
Neshaminy	25	58	36	16	17	33	47
Perkiomen	25	70	42	18	16	31	44
South Fork	11	77	55	27	23	42	57
South Platte	8	98	65	31	26	44	58
Sudbury	22	96	57	24	20	38	54
Tohickon	25	57	36	17	11	21	31
Wachusett	15	82	51	24	19	38	57
Weighted average	293	80.6	49.6	22.0	18.2	35.2	51.7

TABLE 6.—Standard Variations in Storage in Terms of Days' Flow for Several Streams at Various Rates of Draft.

Stream.	Number of years in record.	Standard Variation in Days' Flow.						
		0.20 use.	0.30 use.	0.40 use.	0.50 use.	0.60 use.	0.70 use.	0.80 use.
Bear Creek	11	....	18	21	24	26	31	35
Catskill	17	21	28	32	35	37	38	39
Croton	45	28	38	45	47	48	49	50
Gunpowder	29	9	22	36	40	41	42	45
Hudson	23	....	18	28	32	35	39	43
Manhan	14	20	35	39	43	48	50	53
Merrimac	23	....	15	29	30	35	40	43
Neshaminy	25	35	40	43	44	46	48	50
Perkiomen	25	37	38	39	40	41	43	44
South Fork	11	35	37	46	53	56	58	60
South Platte	8	12	19	26	32	37	40	38
Sudbury	22	16	24	30	34	37	40	42
Tohickon	25	34	35	40	42	44	45	45
Wachusett	15	8	21	28	30	33	36	39
Weighted average	293	24.4	29.0	36.0	38.0	41.1	43.2	45.3

Normal Storage Diagram.—The data in Tables 4 to 7 may be combined to form a composite storage curve representing all the data. The variations in the several cases do not differ widely, but the

absolute quantities do, depending on natural storage and other conditions which are not the same for different streams. The composite curve may be an average or an inclusive one. The latter is selected, and 100 days' storage for a use equal to 50% of the normal flow is taken as the starting point. This covers all but one of the figures in Table 7, and is within the probable error of that one. The exact value taken is of no particular significance, as will appear as the method of use is developed. In a 95% dry year the normal storage (Table 7) will be 68 days more than this, or 168 days. Adding or subtracting the normal differences shown in Tables 4 and 5, to or from 168, gives the figures in Table 8.

TABLE 7.

Stream.	Number of years.	Number of Days' Storage Required to Maintain Flow Equal to 50% of Normal		
		Mean.	95% year.	Difference.
Bear Creek	11	57	100	43
Catskill	17	108	168	60
Croton	45	66	148	82
Gunpowder	29	5	72	67
Hudson	23	36	91	55
Manhan	14	67	142	75
Merrimac	23	45	98	53
Neshaminy	25	62	132	70
South Fork	11	94	186	92
South Platte	8	60	127	67
Sudbury	22	86	144	58
Tohickon	25	91	165	74
Wachusett	15	53	106	53
Weighted average	293	....	....	68

TABLE 8.

Portion which use is of mean flow.	Normal days' storage in average year.	Normal days' storage in 95% dry year.
0.10	(27)	(50)
0.20	49	88
0.30	67	119
0.40	84	145
0.50	100	168
0.60	114	186
0.70	127	203
0.80	139	219
0.90	(150)	(233)
1.00	(160)	(246)

The figures in Table 8 are plotted on probability paper (Fig. 25). The figures for the mean year are plotted on the 54% line, instead of the 50% line, because the curve is a skew curve, and investigation shows

absolute quantities do, depending on natural storage and other conditions which are not the same for different streams. The composite curve may be an average or an inclusive one. The latter is selected,

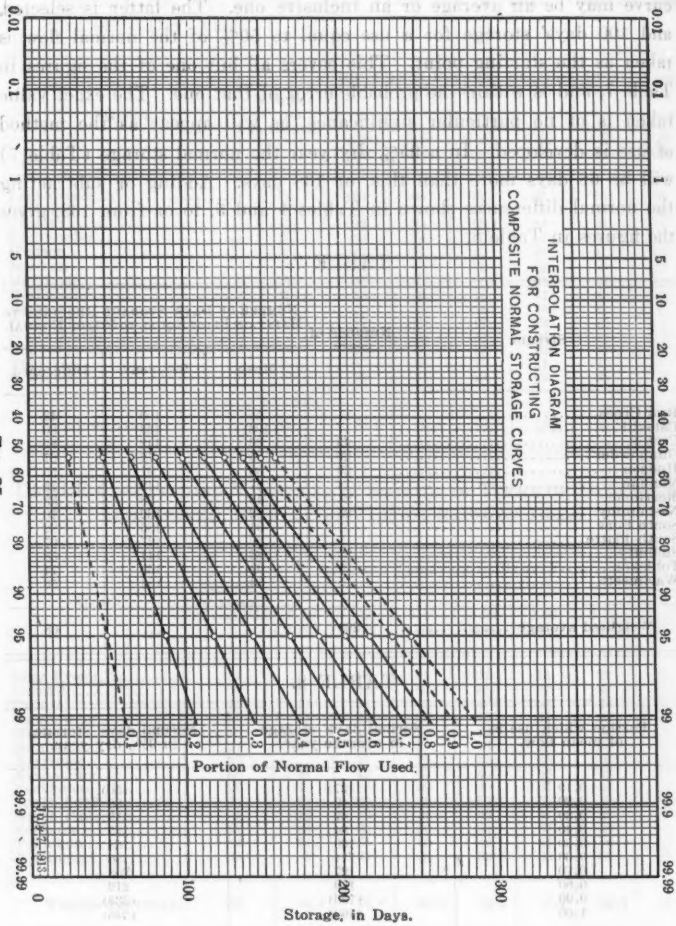


FIG. 25.

The figures in Table 8 are plotted on probability paper (Fig. 25). The figures for the mean year are plotted on the 51% line, instead of the 50% line, because the curve is a skew curve, and investigation shows

that about 54% of all the terms are less than the mean. From this diagram the number of days' storage required to maintain the several assumed rates of flow in years of other degrees of dryness may be taken. Values thus found have been used in plotting Fig. 26, which shows the normal inclusive quantity of storage required to maintain various rates of draft, in years of different degrees of dryness. This diagram, from the method of its construction, is above all but a few individual results.

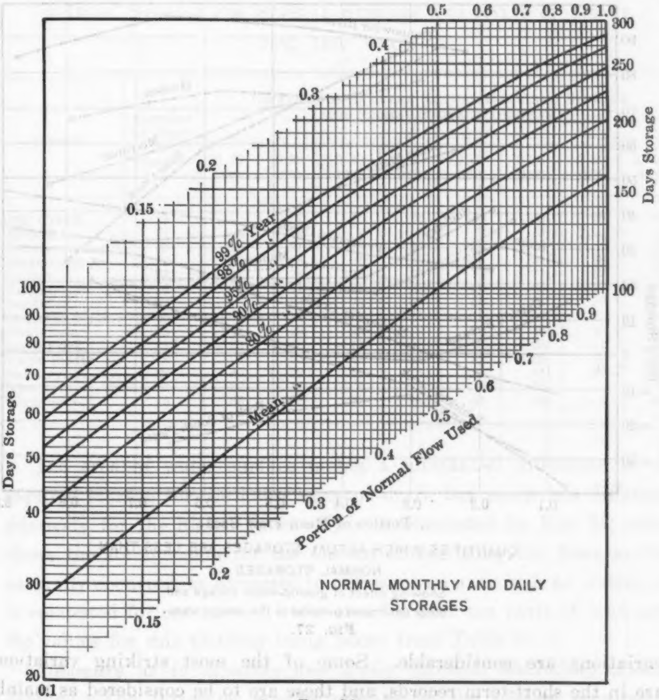
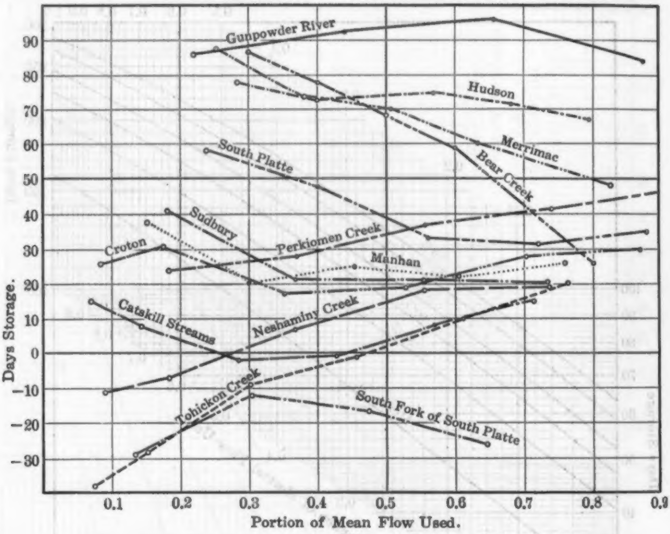


FIG. 26.

The quantity, as shown by the records of various streams, by which the storage actually required falls below the normal storage diagram at various points may be found. Table 9 shows the quantity that they fall below in a 95% dry year, and Fig. 27 shows the difference graphically.

If the normal storage diagram applied strictly to all the records, the figures for Table 9 for each stream at different rates of draft would be constant. This is only approximately the case. The variations represent, in part, accidental variations of the kind that would be found in different parts of a very long record of one stream, and, in part, actual difference in conditions in the different catchment areas, and in their climates and conditions of natural storage, which tend to modify the values of the normal storage diagram. The accidental



QUANTITIES WHICH ACTUAL STORAGES ARE LESS THAN NORMAL STORAGES

Showing effect of ground-water storage and other conditions peculiar to the several areas.

FIG. 27.

variations are considerable. Some of the most striking variations are in the short-term records, and these are to be considered as mainly accidental. The long-term records usually show smaller variations. However, the three Philadelphia streams, Neshaminy, Perkiomen, and Tohickon, show a well-defined tendency to require a greater relative quantity of storage for low rates of draft. This may indicate a greater summer evaporation and a smaller summer run-off, or some variation in seasonal distribution of rainfall. On the other hand, the Merrimac

requires less relative storage for low rates of draft. This may be accounted for in part by the natural lakes on this river, which modify the conditions of run-off by the added evaporation from the water surface, and increase the flow by natural storage in the lakes, which normally runs out gradually during the summer with falling lake level, and tends to maintain the flow.

TABLE 9.—QUANTITY THAT THE ACTUAL MONTHLY STORAGE (CORRECTED FOR DAILY STORAGE) IS LESS THAN THE NORMAL STORAGE DIAGRAM, IN DAYS' STORAGE FOR SEVERAL STREAMS AND RATES OF DRAFT.

95% DRY YEAR.

Stream.	Number of years in record.	Difference Between Actual Required Storage and Normal Storage Diagram.							
		0.20 use.	0.30 use.	0.40 use.	0.50 use.	0.60 use.	0.70 use.	0.80 use.	Average.
Bear Creek	11	....	87	78	68	59	43	26	64
Catskill	17	4	−2	−1	4	9	15	....	5
Croton	45	29	21	18	19	23	28	....	23
Gunpowder	29	85	88	91	93	95	94	88	91
Hudson	23	....	78	73	75	74	71	67	73
Manhan	14	33	21	24	25	22	25	27	25
Merrimac	23	....	84	73	70	63	56	50	66
Neshaminy	25	−6	2	9	15	19	19	....	10
Perkiomen	25	24	26	29	34	38	40	43	33
South Fork	11	−22	−12	−16	−19	−24	−25	....	−20
South Platte	8	....	55	48	39	33	32	33	40
Sudbury	22	40	29	22	21	21	21	20	25
Tohickon	25	−22	−10	−4	2	9	16	....	2
Wachusett	15	63	62	62	62	61	59	....	61

The area of water surface makes a substantial difference in the required storage for the lower rates of draft, but much less difference relatively for the higher rates. This is illustrated by Fig. 28, which shows the storage data for the Croton River, using the flows as they naturally occurred, as corrected to represent an area with no water, and as computed for one part of water area for each ten parts of land area, the values for this plotting being taken from Table 3.

Summary as to Monthly and Daily Storage, Excluding Annual Storage.—The foregoing studies relating to the storage required to maintain flows through one year, and excluding all consideration of cumulative storage, show that, after due allowance is made for the constantly recurring fluctuations, depending on rainfall and other conditions, to analyze which no attempt is made, there remains a fairly definite and simple relation between the draft and the required

storage. There is also a fairly definite and simple relation between the available supplies in years of different degrees of dryness and the frequency of the recurrence of such years. These relations may be expressed in a normal storage diagram which shows the storages in terms of days' draft. The storages thus shown are greater than those actually required on any particular stream by a number of days which is nearly constant for that stream, but varies considerably for different streams, as it depends on the natural storage on the catchment area, and other physical conditions of that stream.

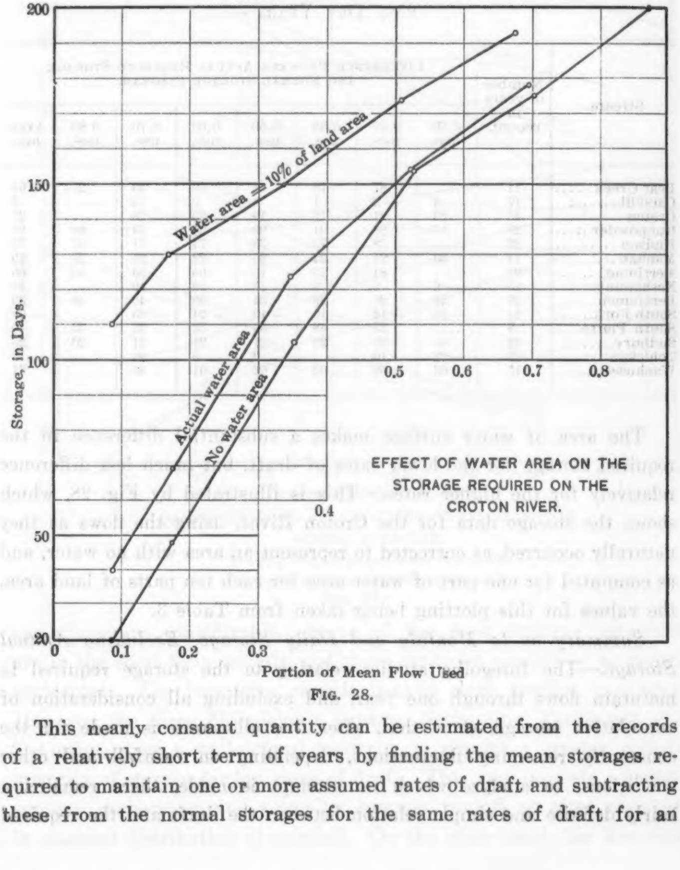


FIG. 28.

This nearly constant quantity can be estimated from the records of a relatively short term of years by finding the mean storages required to maintain one or more assumed rates of draft and subtracting these from the normal storages for the same rates of draft for an

average year. This difference (or the average of several of them) subtracted from the normal storage diagram will give a fair estimate of the storage required under various conditions for that stream, and is probably more accurate than can be made from any but the longest and most carefully kept records of flow for that particular stream.

As to the Application of the Normal Law of Error to Flow and Storage Data.

Discussion of this point has been deferred until the data represented by Figs. 2 to 21, inclusive, could be presented. A study of these diagrams gives an indication of the extent to which the data can be analyzed in this way. If the normal law of error applied strictly, the results for each series would all be found in one straight line. That, of course, is practically impossible. A reasonable approach to a straight line indicates that the data follow the law approximately. In the cases of the longest continued records, the plotted points correspond well with the lines drawn to represent normal conditions. In the records covering shorter periods there are wider fluctuations, owing to the fact that the terms of the series are not numerous enough to have filled in all the intermediate and extreme values that would be found in a long series. In general, the longer the period of record the more closely do the results permit of plotting in a direct line.

To test the matter further, two combination series of results were prepared, showing the variations in terms of the standard variation for each series, first, of the annual flows of each stream for the 300 years, shown in Plate XXXVI; second, the quantities of storage, similarly arranged, required to maintain drafts of 600 000 and 800 000 gal., respectively, per square mile of land area for each of the streams for which these figures were available. A summary of the results of these two series is given in Table 10.

The figures of Table 10 are plotted in Fig. 29. The two series correspond well, that is to say, they have the same degree of skew, and a single line, therefore, represents both. No significance is attached to the fact that they have the same degree of skew. This is considered to be purely accidental. The line representing these results is a direct line, but not a straight one. Most of the results of each series are below the mean; and the variations upward, though less numerous, are greater in magnitude than the variations downward. The results may be combined further, as in Table 11.

TABLE 10.

(1) Actual variation, in terms of the standard variation.	(2) 300 Terms in Series of Annual Flow.		(3) 402 Terms in Series of Required Storage.		(4) Together: 702 Terms.	
	Number below limit in Column 1.	Percentage.	Number below limit in Column 1.	Percentage.	Number below limit in Column 1.	Per-centage.
− 2.5	0	....	1	0.2	1	0.14
− 2.0	0	....	3	0.7	3	0.4
− 1.5	7	2.3	15	3.7	22	3.1
− 1.0	56	18.7	68	17.0	124	17.7
− 0.5	104	34.6	138	34.3	242	34.4
Mean	163	54.3	229	57.1	392	55.8
+ 0.5	218	72.7	275	68.4	493	70.2
+ 1.0	246	82.0	327	81.3	573	81.6
+ 1.5	373	91.0	367	91.2	640	91.2
+ 2.0	288	96.0	391	97.3	679	96.7
+ 2.5	295	98.3	401	99.8	696	99.14
+ 3.0	299	99.7	402	....	701	99.86
+ 3.5	300	100.0	....	....	702	.....

TABLE 11.

Range in standard variations.	Percentage of Terms Outside Range.			
	Below.	Above.	Total.	Computed to follow normal law of error.
0	55.8	44.2	100.0	100.0
0 − 0.5*C*	34.4	29.8	64.2	61.8
0 − 1.0*C*	17.7	18.4	36.1	31.7
0 − 1.5*C*	3.1	8.8	11.9	13.4
0 − 2.0*C*	0.4	3.3	3.7	4.6
0 − 2.5*C*	0.14	0.86	1.00	1.26
0 − 3.0*C*	0.0	0.14	0.14	0.27

In Table 11 the computed and actual variations have been compared. It is seen that the range in combined upward and downward variations agrees as closely as could be expected with the range computed from the normal law of error. The variations upward and downward, taken separately, are not equal. In other words, we have to deal with what is called a skew curve on probability paper. Both ends of this skew curve seem to be nearly straight, with a connecting curve. In drawing the curves for Figs. 2 to 21, inclusive, the ratios in Table 12, obtained from the line in Fig. 29, were used. Straight lines were used to connect these points. No special significance is attached to these figures. Other figures, differing, more or less, from them, would be found from other similar data. The im-

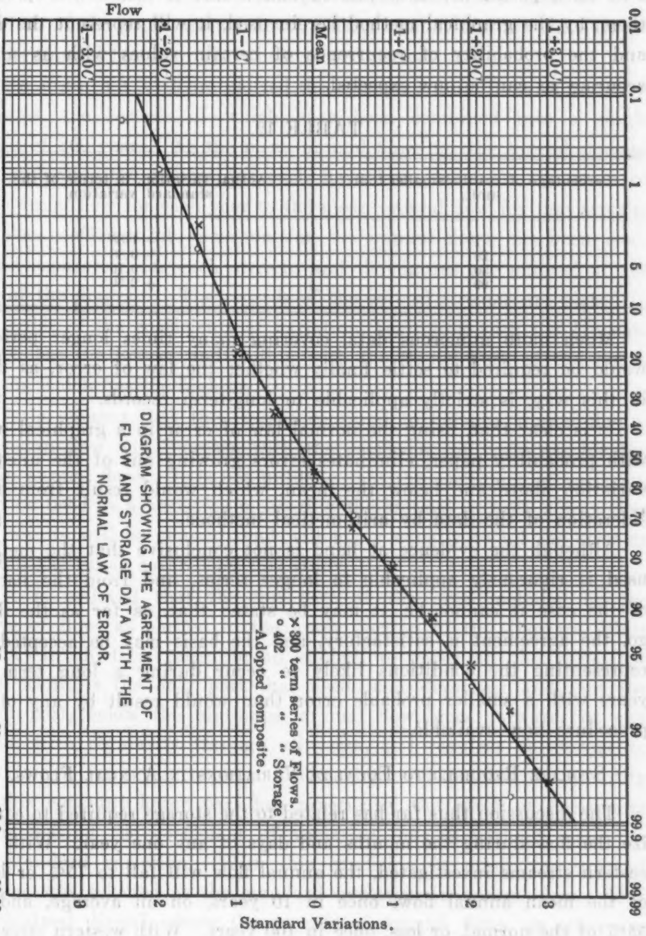

Flow
0.01
0.1
1
5
10
20
30
40
50
60
70
80
90
95
99
99.9
99.99
M−3.0C
M−2.0C
M−C
Mean
M+C
M+2.0C
M+3.0C
DIAGRAM SHOWING THE AGREEMENT OF FLOW AND STORAGE DATA WITH THE NORMAL LAW OF ERROR.
× 300 term series of Flows.
○ 402 " " Storage
—Adopted composite.
3
2
1
0
1
2
3
Standard Variations.

FIG. 29.

portant point is that data of this kind plotted on probability paper are close to a line which is direct and nearly straight at the ends, and when points are thus plotted and a line is drawn to represent them, by the graphical method herein used, it will represent the data and the probability of recurrence of certain values with as much accuracy as can be now expected.

TABLE 12.

Percentage of results smaller than limit.	Actual variation, in terms of the standard variation.
1	−1.83
20	−0.88
50	−0.10
99	+2.50

Much more numerous data, covering many times longer periods, would be required to settle finally whether the law of error, as used in this way, is strictly applicable to long-term records.

It is clear that, using the normal law of error in a graphical way, with probability paper, eliminates errors growing out of the unequal variation above and below the mean, which would result from consideration of the data by arithmetical methods.

Although the evidence at hand is not conclusive that the method used is rigorously applicable to longer terms, and from the nature of the case it cannot be, it may be stated that, as far as the data go, the agreement is satisfactory, and the basis may be accepted as representing the conditions likely to occur during a long term of years with a smaller probable error than would result by any other procedure now available.

STORAGE REQUIRED TO EQUALIZE VARIATIONS IN ANNUAL FLOWS.

The discussion thus far has related to the storage required to equalize the flow during the months and days of any one year. With the eastern streams investigated, the annual flow will fall to 75% or less, of the mean annual flow, once in 10 years, on an average, and to 55% of the normal, or less, once in 100 years. With western streams, the range is greater. It is possible, by storage, to use more water at all times than flows in a dry year, but to do this it is necessary to carry water over from wet years to make it available in dry ones.

In the following paragraphs "annual flow" means the average of observations during a period of one year, stated either in millions of gallons per day, or cubic feet per second per square mile, or as a fraction of the mean annual flow. "Mean annual flow" is the average of the annual flows for the whole record period.

On Plate XXXVI are plotted the relative annual flows of fourteen streams, the records of which have been combined into a single series of 300 years. As the mean annual flows for the several streams are different, the figures for each have been taken as the ratio of the annual flow for each year to the mean annual flow for that stream.

The coefficient of variation for the annual flows has been computed separately for each stream, and the figures are entered at the top of the diagram. The records of the several streams are placed in the order of the coefficients of variation, beginning with the Hudson, which has the lowest, and ending with the South Fork of the South Platte, which has the highest.

It is obvious that the storage required to balance annual fluctuations in flow will increase with the coefficient of variation for the mean annual flows. It is not unlikely that the coefficient of variation may be used as a basis for measuring the storage in such a way that the results will be general, and will apply equally to all the streams within the range covered by this study.

It is believed that the top part of the area included by the curve of flow, as plotted in the lower portion of the diagram, Plate XXXVI, which is taken broad enough to cover all the variations that occur, is the only part that needs consideration; and that the lower part, below the minimum annual flow, goes forward, in any event, with regularity through all years, and requires no storage to maintain it from year to year. Increasing or decreasing the quantity of this constant lower part has practically the effect of decreasing or increasing the coefficient of variation, but is without effect on the standard variation. With this condition in mind, the unit of storage was selected as the standard variation in annual flow. This is obtained directly from the records of annual flows, or otherwise by multiplying the mean annual flow by the coefficient of variation.

The rate of draft must also be expressed in a form in which the coefficient of variation will be an element. The method selected was

to compute the storage required for drafts equal to the mean annual flow, less a certain part of a "standard variation". These may be represented conveniently by the formula:

$$\text{Mean Annual Flow} \times [1 - k \times (\text{Coefficient of Variation})].$$

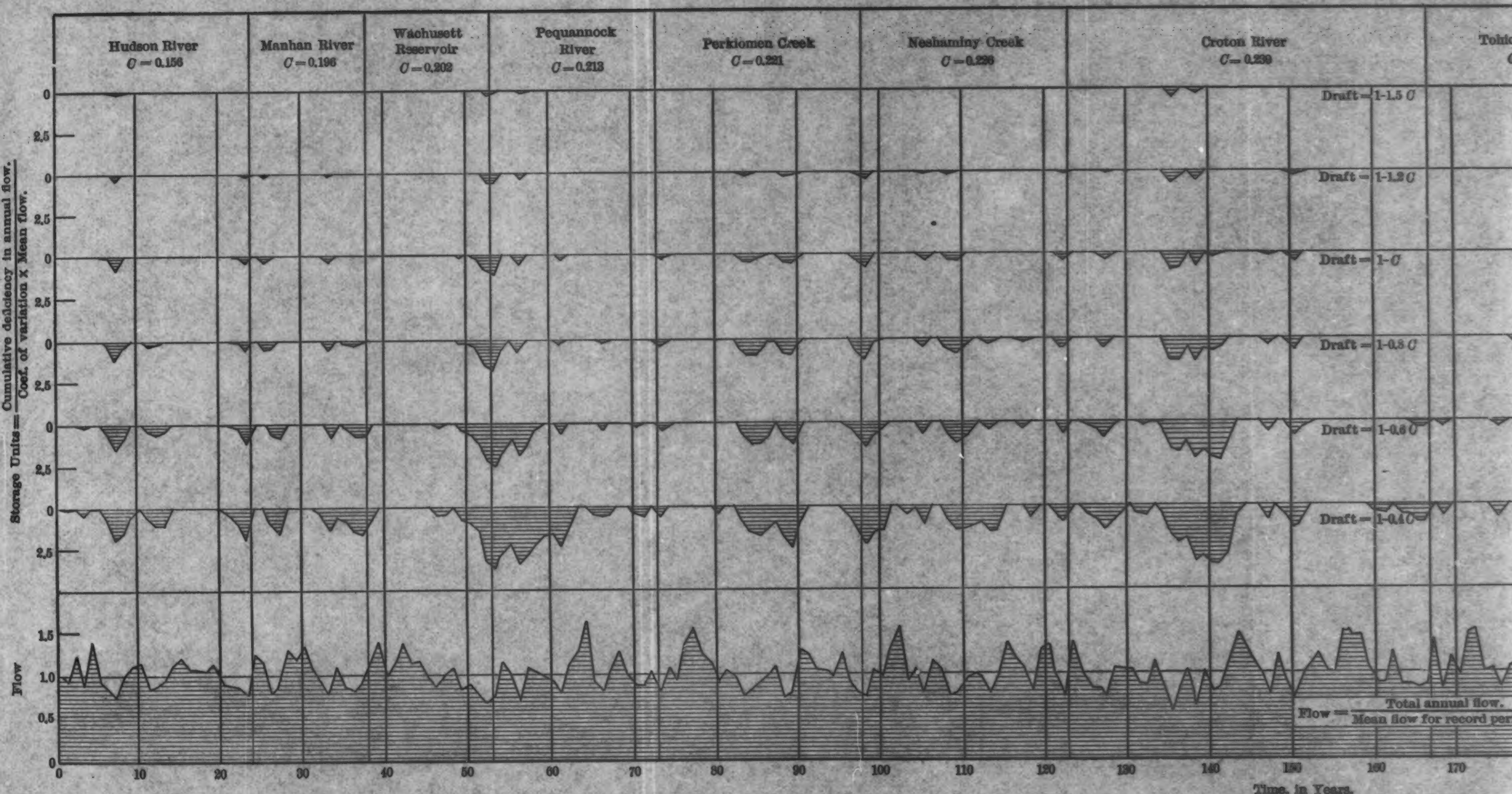
Method of Calculating Annual Storage.—The calculation is carried out by assuming an indefinitely large reservoir, full at the beginning of the record period, with a constant assumed rate of draft from it, and finding the depletion, if any, at the end of the first year, with the flows occurring as shown by the records, and at the end of the second year, and of every year for the whole period. In carrying this out, monthly flows are disregarded; only mean annual flows are taken into account.

The rate of draft corresponding to the mean annual flow less 0.4, 0.6, 0.8, 1.0, 1.2, and 1.5 standard variation in annual flow, and also the value of one unit of storage, is calculated for each stream separately, and these are used in examining the terms in the series derived from the record of that stream. The required storage, that is to say, the computed depletion of an indefinitely large reservoir at the end of each year, is then stated in units of storage. When deficiency of storage is indicated at the end of any of the subdivisions of the whole series corresponding to the records of one stream, it is carried forward into the record of the following stream until the reservoir would have refilled.

The quantities of storage, computed in this way for each year in the 300-year series for each of the six assumed relative rates of draft, are shown graphically on Plate XXXVI. This plotting gives a good idea of the periods and of the relative quantities of depletion at the several rates of draft in different parts of the whole series, but it is on too small a scale to be used as a basis of further calculation, and the actual figures from which it was made are used for that purpose.

It is interesting to note that the dry periods on the Sudbury and on the Croton River, when computed in this way, show substantially equal quantities of depletion, but that the Wachusett and Pequannock records, taken together, show a period of depletion fully equal to these, and the Gunpowder and South Platte records, taken together, a depletion only slightly less, thus indicating, as far as these data go, that storages as great relatively as those required at the dry

DISTRIBUTION OF ANNUAL STORAGE PERIODS.



DISTRIBUTION OF ANNUAL STORAGE PERIODS.

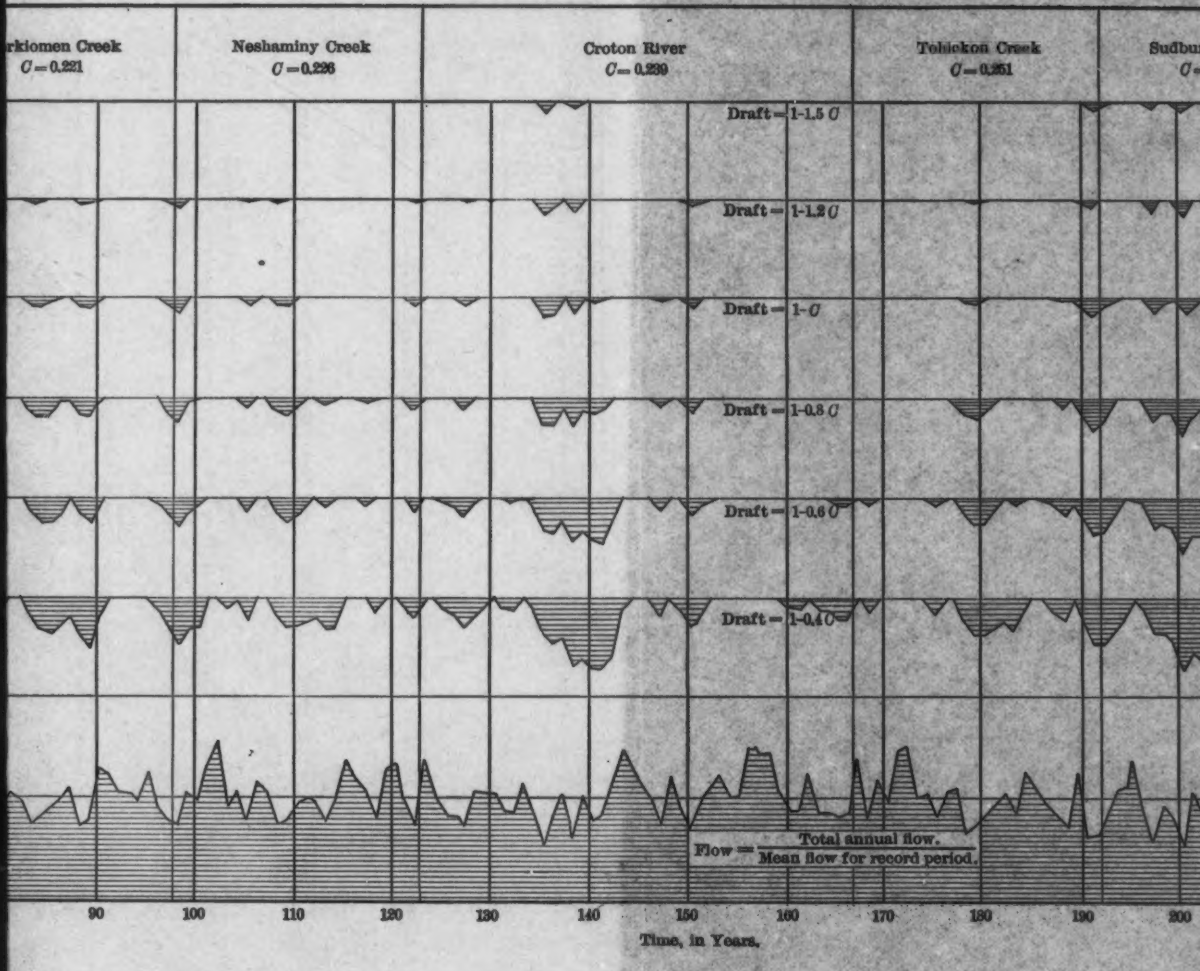
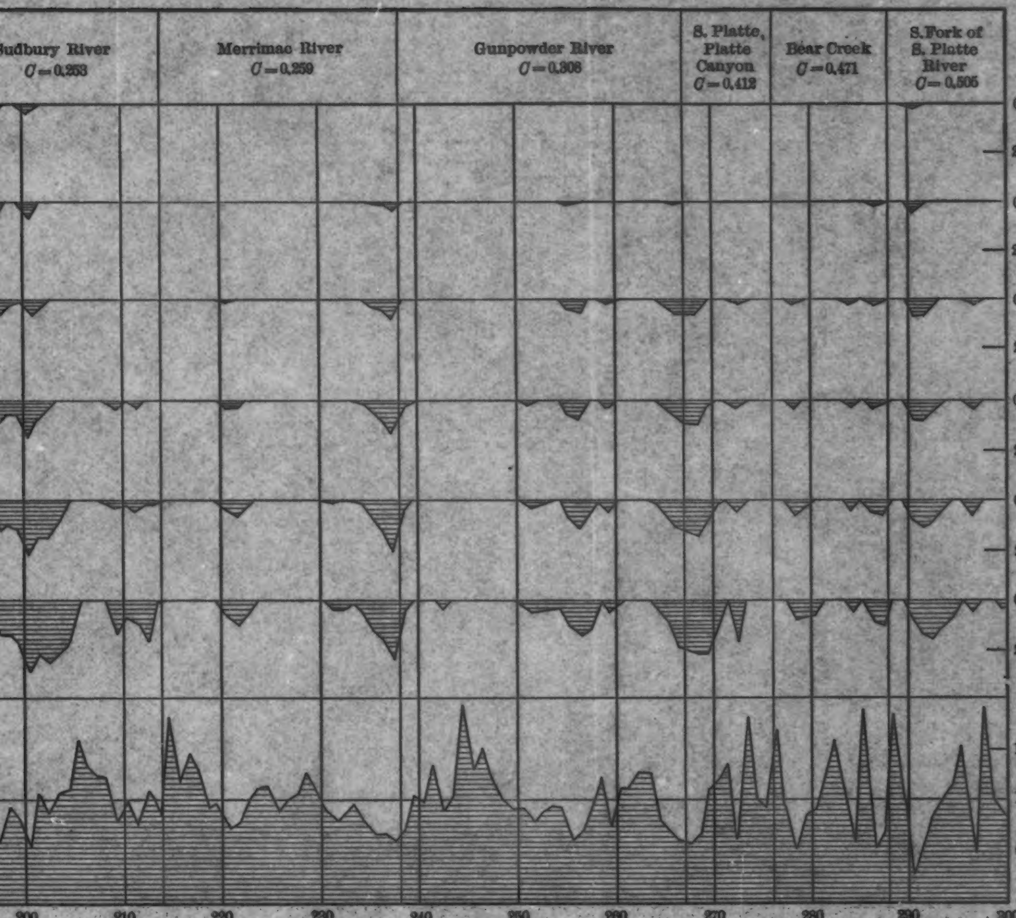


PLATE XXXVI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVII, No. 1308.
HAZEN ON
STORAGE TO BE PROVIDED
IN IMPOUNDING RESERVOIRS.



times about 1880 for the Croton and Sudbury may be expected to recur from time to time in other streams.

The plottings also indicate that the method of bringing the coefficient of variation into the calculation has practically accomplished the desired purpose of arranging all the records so that the computed number of storage units do not differ, or at least do not differ widely, for streams having high coefficients of variation, and for those having low ones. In other words, the depletion for the various relative drafts is about as high at one end of the diagram as at the other. If the adjustments were not fairly well made, there would be an excess of storage indicated at one of the ends.

On Plate XXXVII are plotted, for each of the six rates of draft, the figures representing the cumulative number of storage units at the end of each of the 300 years, or as many of them as show depletion, arranged in the order of their magnitude, and lines have been drawn to represent, as nearly as possible, the normals for each series. This plotting is made on probability paper. The lines drawn to represent the data are straight, and the deviations of individual points from them are small. From this diagram the values in Table 13 are taken.

TABLE 13.—Annual Storage, in Units of Standard Variation in Annual Flow, Required to Balance Annual Fluctuations in Stream Flow with Various Rates of Draft.

Relative rate of draft.	80% dry year.	90% dry year.	95% dry year.	98% dry year.	99% dry year.
$1 - 0.4C$	1.40	2.01	2.52	3.10	3.48
$1 - 0.6C$	0.75	1.21	1.59	2.02	2.31
$1 - 0.8C$	0.30	0.65	0.95	1.28	1.49
$1 - 1.0C$	0.04	0.31	0.55	0.81	0.97
$1 - 1.2C$	....	0.07	0.27	0.47	0.62
$1 - 1.5C$	....	.....	0.00	0.19	0.31

In Table 14 the coefficient of variation of mean annual flow has been determined from certain other streams for which run-off records are available, for the purpose of getting a somewhat broader basis for forming a judgment of the probable coefficient of variation for other streams. No effort has been made to secure completeness in this table, which has only been extended to cover certain data readily available, mostly from records of the U. S. Geological Survey.

Final Arrangement of Annual Storage.—The data of Table 13 have been replotted in Fig. 30, lines being drawn for years of each of

the five degrees of dryness used in the calculation, showing the quantities of annual depletion to be anticipated, plotted on a base of the relative rates of draft. This diagram can be used conveniently in computing the probable annual storage required for a given stream.

TABLE 14.

Stream.	Catchment area, in square miles.	Years in record period.	Coefficient of variation in annual flow.
Kennebec River, Waterville, Me	4 270	19	0.26
Androscoggin River, Rumford Falls, Me	2 220	17	0.16
Cobbosseecontee Pond, Gardiner, Me	240	21	0.25
Mystic Lake, Mass	27	20	0.28
Connecticut River, Hartford, Conn	10 234	13	0.15
Connecticut River, Holyoke, Mass	8 660	19	0.21
Passaic River, Dundee Dam	822	17	0.21
Susquehanna River, Harrisburg, Pa	24 000	20	0.16
Potomac River, Point of Rocks, Md	9 650	15	0.33
Yadkin River, Salisbury, N. C	......	14	0.27
Ocmulgee River, Macon, Ga	2 425	18	0.26
Savannah River, Augusta, Ga	7 294	17	0.25
Ohio River, Wheeling, W. Va	23 800	22	0.19
Tennessee River, Chattanooga, Tenn	22 418	21	0.20
Kansas River, Lawrence, Kans	58 550	14	0.65
Republican River, Junction, Kans	25 837	9	0.61
Arkansas River, Cañon, Colo	3 060	19	0.23
Rio Grande, Del Norte, Colo	1 400	16	0.34
Bear River, Collinston, Utah	6 000	11	0.23
Provo, Provo Cañon, Utah	640	16	0.22
Mill Creek, Salt Lake City, Utah	21	13	0.35
Parley's Creek, Salt Lake City, Utah	50	11	0.58
City Creek, Salt Lake City, Utah	19	10	0.34
Humboldt River, Elko, Nev	2 840	14	0.41
Tuolumne River, La Grange, Colo	1 501	16	0.41
Columbia River, Dalles, Ore	237 000	32	0.20
Willamette River, Albany, Ore	4 860	17	0.26
Sweetwater Dam, California	....	24	1.37

It may be noted that the expression, "95% dry year", as used herein, does not refer to any particular year. Its use means that, with a given rate of draft and unlimited storage, there would probably be 5 years in 100 when the depletion of storage would exceed the limit shown.

It is probable that 2 or more of the 5 years would follow consecutively in one period of several years of low average flow.

Combined Results for Monthly and Annual Storage.

It is now possible to construct a diagram showing the storage required to maintain various rates of draft for years of different degrees of dryness for a stream for which the mean annual flow, the coefficient of variation in annual flows, and the ground-water storage are known. Fig. 31 shows the Croton data arranged in this way, and

PLATE XXXVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVII, No. 1308.
HAZEN ON
STORAGE TO BE PROVIDED
IN IMPOUNDING RESERVOIRS.

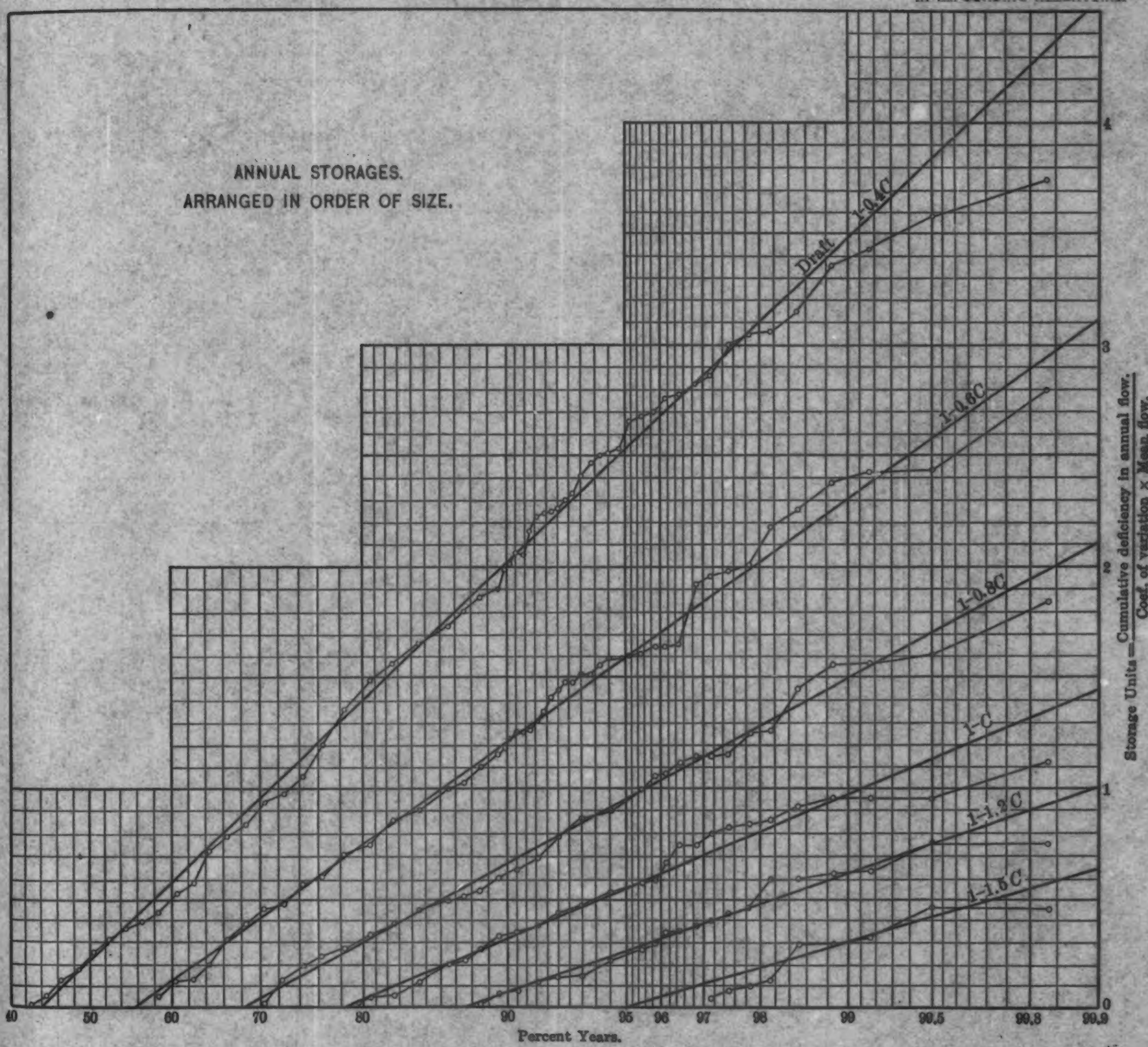


Fig. 32 shows the Sudbury data. Table 15 shows the method by which the figures from which the diagrams are made were obtained.

TABLE 15.—CROTON STORAGE.—95% YEAR.

Mean annual flow, 1 sq. mile land area, 1 137 000 gal. daily.

Coefficient of variation in mean annual flows, 0.239.

Constant deduction for ground-water storage, etc., Croton River, as in Table 9, 23 days.

Storage unit, 1 137 000 × 365 × 0.239 = 99 500 000 gal.

Draft, in gallons per day per square mile of land area.	Draft divided by mean flow.	Monthly and Daily Storage.			Annual Storage.			Total storage required.
		Days' storage required; normal diagram.	Days' storage less 23 for natural storage.	Storage, in millions of gallons per square mile.	Relative rate of draft.	Units of storage required.	Millions of gallons per square mile of land area.	
1 028 000	0.904	233	210	216	$1-0.4C$	2.52	251	467
973 000	0.856	227	204	198	$1-0.6C$	1.59	158	356
918 000	0.808	220	197	181	$1-0.8C$	0.95	95	276
866 000	0.761	213	190	164	$1-1.0C$	0.55	55	219
811 000	0.713	206	183	148	$1-1.2C$	0.27	27	175
729 000	0.641	193	170	124	$1-1.5C$	0.00	0	124
650 000	0.572	181	158	103	......	......	....	103
600 000	0.528	172	149	89	......	......	....	89
500 000	0.440	154	131	65	......	......	....	65
400 000	0.352	133	110	44	......	......	....	44
300 000	0.264	109	86	26	......	......	....	26
200 000	0.176	80	57	11	......	......	....	11

Using the diagram, Fig. 33, the construction of which will be explained subsequently, the foregoing calculation is more easily made, with the same results, as shown in Table 16.

TABLE 16.

Draft, in millions of gallons per day per square mile of land area.	Draft divided by mean flow.	Days' storage, 95%-year storage diagram.	Days' storage, less 23 for natural storage.	Days' total storage required per square mile of land area.
1.028	0.904	478	455	468
0.973	0.856	385	362	352
0.918	0.808	322	299	274
0.866	0.761	278	255	221
0.811	0.713	240	217	176
0.729	0.641	195	172	125
0.650	0.572	182	159	103
0.600	0.528	173	150	90
0.500	0.440	154	131	66
0.400	0.352	133	110	44
0.300	0.264	109	86	26

On Figs. 31 and 32 two sets of lines have been drawn. The first are solid, and represent figures computed from the normal storage curve with a deduction of the constant found for that stream, which for the Croton is 23 days and for the Sudbury, 25 days. The second set, dotted lines, and the figures from which they are plotted, were scaled from Figs. 8 and 19, with the addition of 9 days for daily storage. In

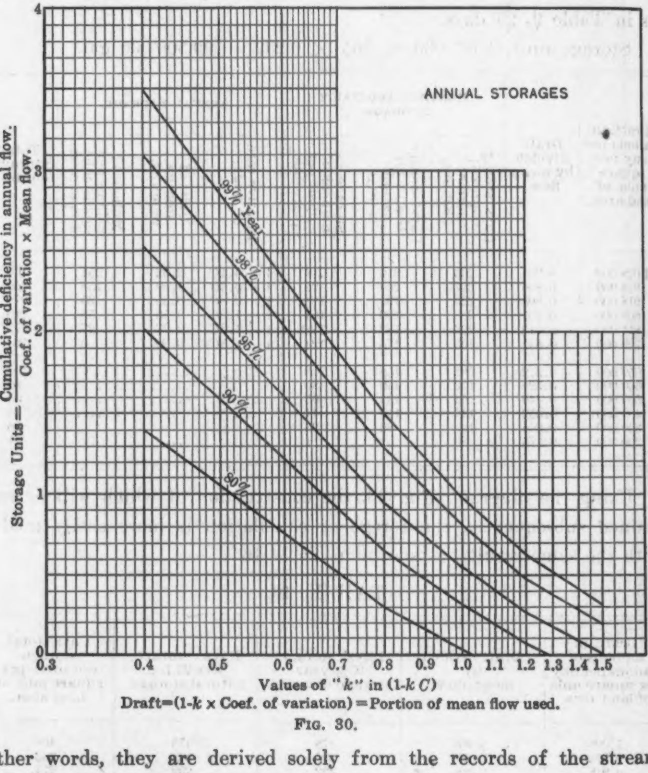


FIG. 30.

other words, they are derived solely from the records of the stream itself, and the normal diagram is not used. The allowances for annual storage were made, in all cases, from the figures in Table 13. Where the two sets of lines were practically in the same position, the dotted one was not drawn. For both the Sudbury and Croton, for the higher rates of draft, which are practically the most important ones, the annual storage

dominates, and it makes no appreciable difference which of the two methods of calculation is used. For the lower rates of draft, there are deviations. In the Croton these deviations are small; in the Sudbury they are greater. In the latter the lines showing the actual records for years of different degrees of dryness are somewhat closer to one another than those deduced from the normal diagram, and with the Croton they are farther apart.

It is a matter of debate whether the lines obtained from the normal storage curve, or from the records of the stream itself, best represent the conditions on a particular stream. On the Croton, with its longer record, it makes but little difference. On the Sudbury, with a shorter record, the divergencies are greater. The writer feels that the lines obtained from the normal storage curve are more reliable than those from this relatively short-record period.

Critical Storage.—The point in the storage curve which marks the beginning of the use of storage to carry water over from one year to another is characterized by a well-marked angle. As many of the phenomena of storage differ somewhat, below and above this point, it will be convenient to give this angle a name, and it will be called the "critical point".

The values for storage found and used by Messrs. FitzGerald, Stearns, and Freeman, and stated in their several publications, are plotted on these diagrams for comparison with those calculated at this time. In a general way, for values below the critical point, they correspond to the 98% dry year; above that point they correspond to the 95% dry year. In other words, they represent conditions which would be expected to recur not oftener than once in 50 years and 20 years, respectively.

There is a particularly interesting case in connection with the Croton, where the storage, as first computed by Mr. Freeman, for drafts of 900 000 and 1 000 000 gal. per day per sq. mile, gave results which, as plotted on Fig. 31, come between the 90 and 95% years. Mr. Freeman showed, by an analysis of the conditions, that an extraordinary rain occurred in the middle of what was otherwise the dryest period. He considered that there was no certainty of like rains occurring coincidently with other similarly dry periods, and made certain allowances on the basis that the quantity of rain that actually fell might have been less. He concluded that certain larger values

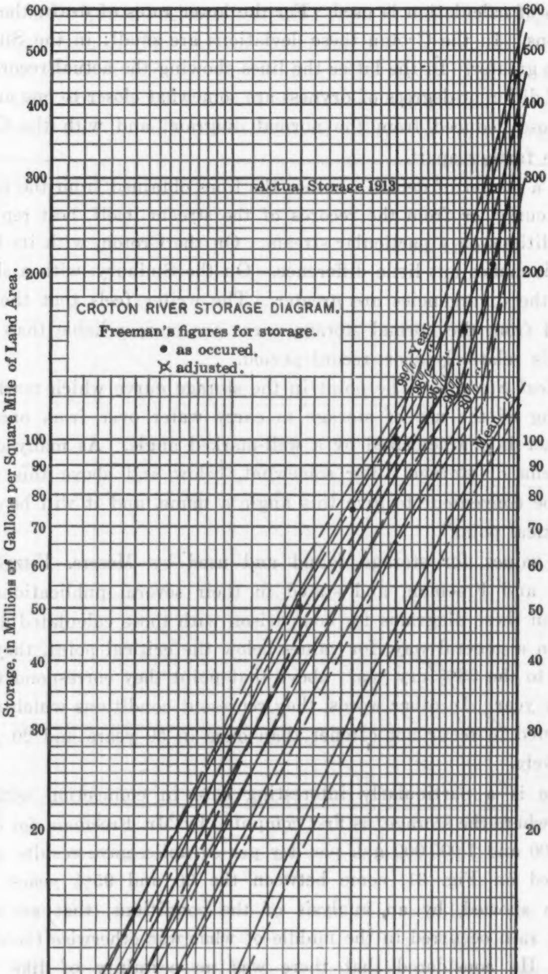

CROTON RIVER STORAGE DIAGRAM.
Freeman's figures for storage.
• as occured.
✕ adjusted.
Actual Storage 1913
99% Year
Mean
Storage, in Millions of Gallons per Square Mile of Land Area.
Draft, in Millions of Gallons Daily per Square Mile of Land Area.
600
500
400
300
200
100
90
80
70
60
50
40
30
20
10
0
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

FIG. 31.

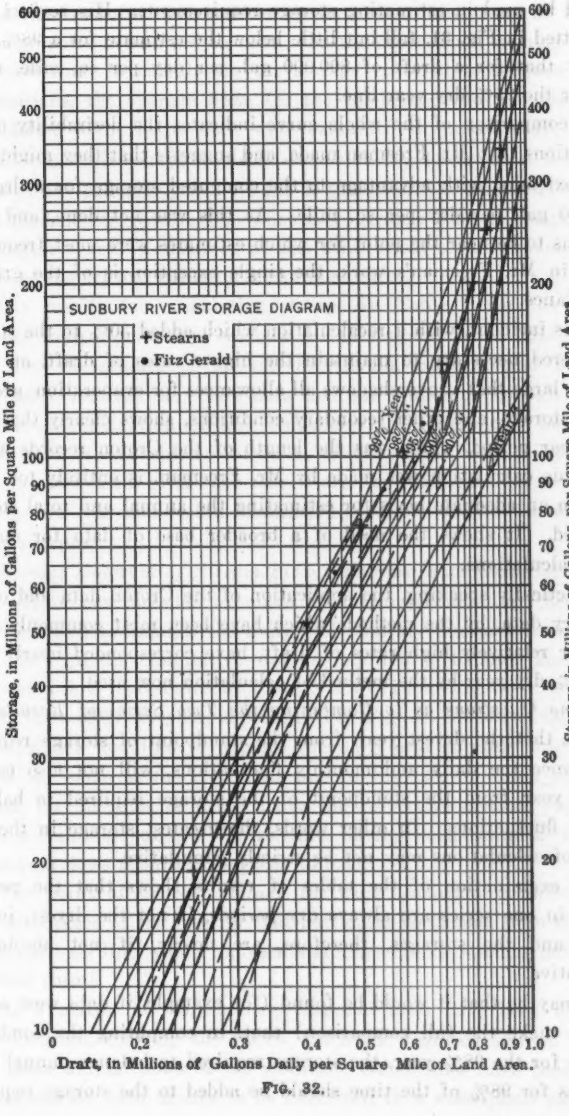

SUDBURY RIVER STORAGE DIAGRAM
+ Stearns
• FitzGerald
Storage, in Millions of Gallons per Square Mile of Land Area.
Draft, in Millions of Gallons Daily per Square Mile of Land Area.
600
500
400
300
200
100
90
80
70
60
50
40
30
20
10
0
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

FIG. 32.

should be used in estimating storage requirements. His revised data, as plotted in Fig. 31, fall but little below the estimate for a 98% year, except that for a draft of 800 000 gal. per day per sq. mile, which is near the 95% dry year line.

A comparison of the whole curve indicates the desirability of the corrections that Mr. Freeman made, and suggests that they might have been extended with advantage to the computed storage for a draft of 800 000 gal. per day per sq. mile. As this was not done, and as it happens to be near the point for which estimates were most frequently made in Mr. Freeman's work, the single exception is of the greatest importance.

This incident, with a recalculation which added 30% to the storage considered necessary to maintain the higher rates of draft, an addition so large that it overshadows all allowances for evaporation, ground-water storage, and other secondary conditions, shows clearly that even a 30-year period, which was the length of the Croton records at the time this calculation was made by Mr. Freeman, is entirely too short to form an adequate basis for estimating the annual and total storage required. It shows the need of a broader base of data for making such calculations.

Practically speaking, the application of the Croton data and of the Sudbury data, by the methods which have been most commonly used, and for relatively high rates of draft, have corresponded nearly with the 95% dry year by the method of calculation now used.

Some Questions as to Combining the Two Series of Results.—It may be that the dryest year, from the standpoint of storage required to balance the daily and monthly fluctuations, will not also be the dryest year from the standpoint of the storage required to balance annual fluctuations. In other words, the greatest storage in the two series of calculations may not be strictly cumulative.

An examination of the tables of results shows that the periods dryest in one series are always dry periods, if not the dryest, in the other, and the storages, therefore, are nearly, if not absolutely, cumulative.

It may be that it would be found (for example, if data were available to make the full comparison) that, in computing the combined storage for the 98% year, the storage required to balance annual fluctuations for 98% of the time should be added to the storage required

to balance monthly fluctuations up to 90% of the time, or to some other proportion than 98 per cent. On the other hand, it must be kept in mind that, in calculating the storage required to balance annual fluctuations, the average flows for the calendar years only have been considered, and, if the data were examined in more detail, periods of 365 consecutive days would be found having lower mean flows than those of any calendar year. So far as this condition exists, there would be a tendency to require somewhat more storage than is computed for the dryest years. Conditions of this kind would tend to round off the angle at the critical point in the storage curve.

The results for cumulative storage in Table 1 for the various streams were examined to see how far they would throw light on this, and also the studies of Messrs. FitzGerald, Stearns, and Freeman, previously referred to. The data are not numerous enough to give a clear-cut indication of any tendencies that there may be at this point, and, in the absence of such indications, it may be considered that the two series of storage results are, for practical purposes, cumulative, and that the results obtained by adding one to the other are to be accepted.

Normal Diagram of Stream Flow Combined with Cumulative Storage.—It is now possible to make a diagram, combining the normal flow required to balance the daily and monthly fluctuations shown in Fig. 26 with the cumulative storage required to balance fluctuations in annual flow represented by Plate XXXVII and Fig. 30. Such a diagram, for a 95% dry year, is presented in Fig. 33. It shows the storage in days' flow required to balance the daily and monthly fluctuations, being the same as the 95% year line in Fig. 26, to which have been added, for all rates of draft above the respective critical points, the required annual storages for streams having coefficients of variation in annual flows ranging from 0.05 to 0.60.

The point of critical storage for each condition is indicated by the divergence of the line for that particular coefficient from the heavy diagonal base line which represents storage for all streams below their critical points.

As a matter of convenience, diagonal lines have been drawn on this diagram, showing the ratios which the required storage bears to the mean annual flows of the streams. Other lines have been drawn, to correspond with figures to be developed in a subsequent paragraph,

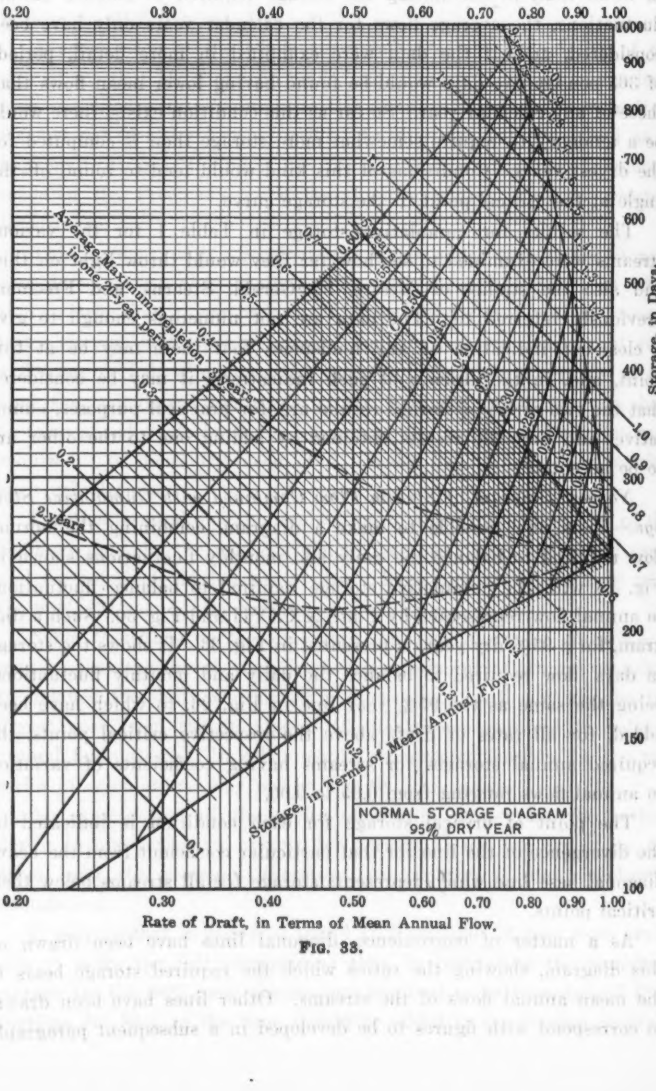

0.20 0.30 0.40 0.50 0.60 0.70 0.80 0.90 1.00
1000
900
800
700
600
500
400
300
200
150
100
Storage, in Days.
Average Maximum Depletion in one 20-year period.
2 years
3 years
5 years
9 years
Storage, in Terms of Mean Annual Flow.
NORMAL STORAGE DIAGRAM
95% DRY YEAR
Rate of Draft, in Terms of Mean Annual Flow.

Fig. 33.

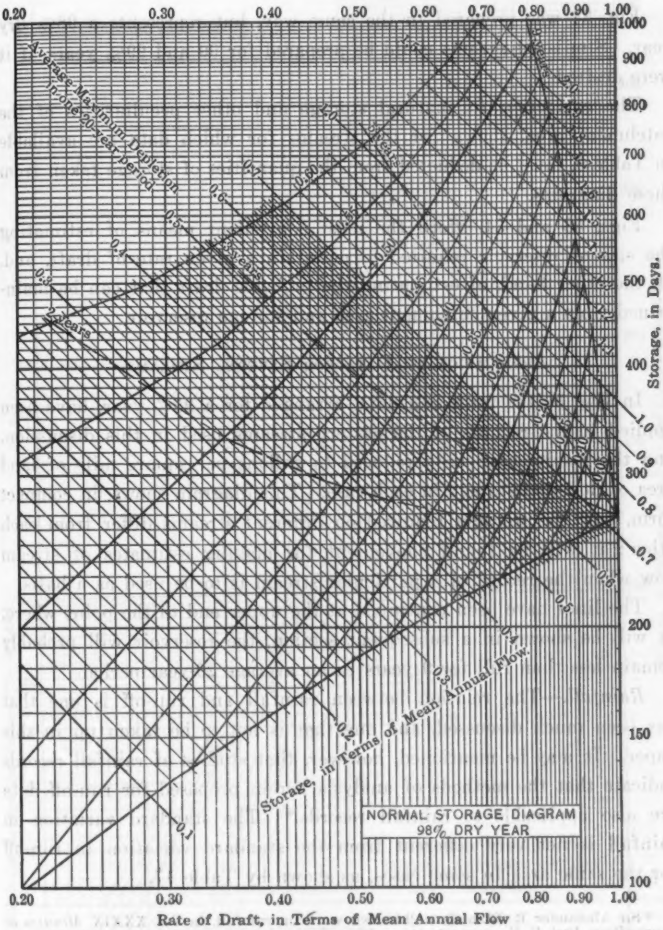

0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
1000
900
800
700
600
500
400
300
200
150
100
Storage, in Days.
Average Maximum Depletion in one 20-Year period
Storage, in Terms of Mean Annual Flow.
NORMAL STORAGE DIAGRAM
98% DRY YEAR
Rate of Draft, in Terms of Mean Annual Flow

Fig. 34.

showing the points at which the reservoir will probably be less than full for periods of 2, 3, 5, and 9 years, respectively, in an average 20-year period.

Fig. 34 was prepared in the same way, but represents a 98% dry year. Similar diagrams could be prepared for 90 and 99% years, if it were desired.

The correction for natural storage and other peculiarities of the catchment area, shown for the streams for which data are available in Table 9, must be deducted from the quantities of storage taken from these diagrams.

Figs. 33 and 34 afford the most convenient means of estimating the size of reservoir required to maintain a given rate of draft, and, on the other hand, of estimating the probable draft that can be maintained from a given catchment area and storage reservoir.

Application to Data for Several Streams.

In Fig. 35 the foregoing data, arranged for a 95% year, have been applied to the records of the eastern streams used in this discussion, and the results have been reduced to gallons per square mile of land area of flow and storage in each case. This diagram shows, in compact form, how the storages required on different streams differ from each other; it will be found convenient for making estimates of stream flow when the record of a particular stream is to be used as a basis.

The lines have been continued at the upper end to the point where, as will be shown in a subsequent section, the reservoir will probably remain less than full for 9 years in an average 20-year period.

Rainfall.—The relation between rainfall and run-off is one that has been much discussed, and one that is not to be taken up in this paper. It may be mentioned, however, that studies of rainfall records indicate that the methods of analysis herein proposed for run-off data are also applicable to rainfall records.* The standard variation in rainfall is not very different from the standard variation in run-off for the same area in some cases, as shown by Table 17.

* Sir Alexander R. Binnie's well-known work on rainfall, in Vol. XXXIX, *Minutes of Proceedings*, Inst. C. E., suggests this, although the data were not arranged in the way now proposed. The late George W. Rafter, M. Am. Soc. C. E., suggested the application of the method of least squares to rainfall data in the report of the New York State Engineer for 1896, with a view to deducing the probability of occurrence of certain conditions of low flow, and announced the intention to discuss in some future report the final theoretical question of methods of deducing a formula to enable one to take the rainfall and temperature records of a long series of years and deduce therefrom the probable yearly run-off of a stream.

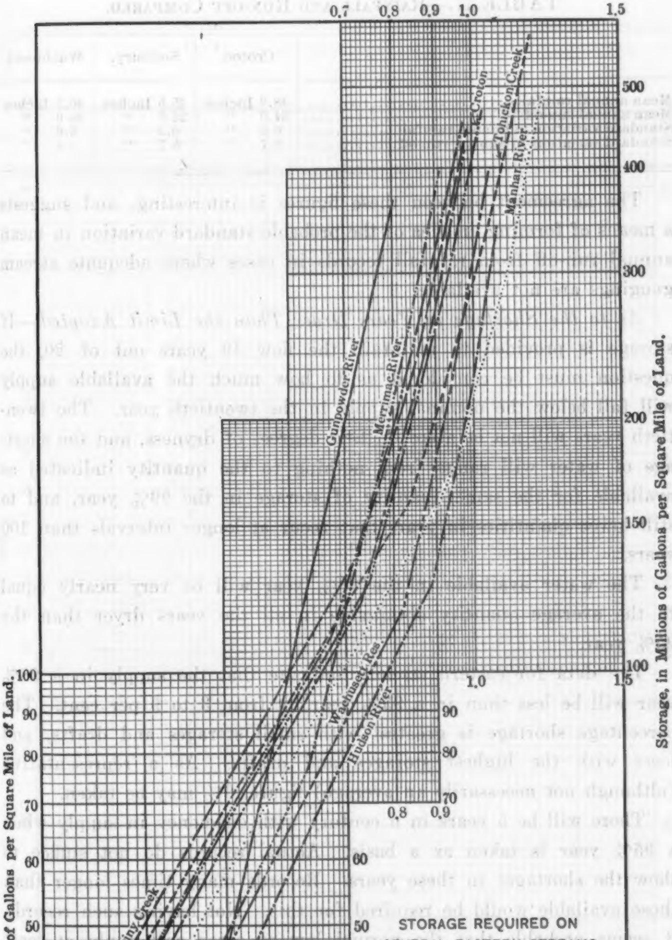

STORAGE REQUIRED ON
SEVERAL EASTERN STREAMS
95% YEAR
Gunpowder River
Merrimac River
Croton
Tohickon Creek
Mahanar River
Wachusett Res.
Hudson River
Neshaminy Creek
Perkiomen Creek
Sudbury River
Storage, in Millions of Gallons per Square Mile of Land.
Draft, in Millions of Gallons per Day per Square Mile of Land.
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.5
40
50
60
70
80
90
100
150
200
300
400
500

FIG. 35.

TABLE 17.—RAINFALL AND RUN-OFF COMPARED.

	Croton.	Sudbury.	Wachusett.
Mean annual rainfall	48.8 Inches	45.5 Inches	46.5 Inches
Mean annual run-off	24.0 "	22.9 "	24.0 "
Standard variation in annual rainfall	6.0 "	6.5 "	6.6 "
Standard variation in annual run-off	5.7 "	5.7 "	4.8 "

The agreement between these figures is interesting, and suggests a means of forming an idea of the probable standard variation in mean annual run-off from rainfall records in cases where adequate stream gaugings are not available.

As to the Shortage in Years Dryer Than the Limit Adopted.—If storage is provided to maintain the flow 19 years out of 20, the question must be considered as to how much the available supply will fall below the nominal supply in the twentieth year. The twentieth years will not be alike in their degrees of dryness, and the shortage of water will range from nothing to the quantity indicated as available for the same quantity of storage in the 99% year, and to still lower quantities in years that recur at longer intervals than 100 years.

The water available in the 98% year will be very nearly equal to the average quantity obtainable in all the years dryer than the 95% year.

The data for eastern streams indicate that the supply in a 98% year will be less than in a 95% year by from 3 to 9 per cent. The percentage shortage is greatest with small storages and drafts, and least with the highest storages and drafts. As a representative (although not necessarily an average) figure, 6% may be taken.

There will be 5 years in a century with deficiency in supply when a 95% year is taken as a basis. Actual records do not suffice to show the shortages in these years. Records many times longer than those available would be required for this. Not having such records, it seems probable that the normal law of error will apply at least approximately, and that probability paper may be used for a graphical solution of this problem. In Fig. 36 the basis of such a calculation is indicated. A line is drawn passing through 100% in the 95% year, and 94% in the 98% year, and this is continued. Values taken from it indicate that the probable shortage in the 5 dry years of a century

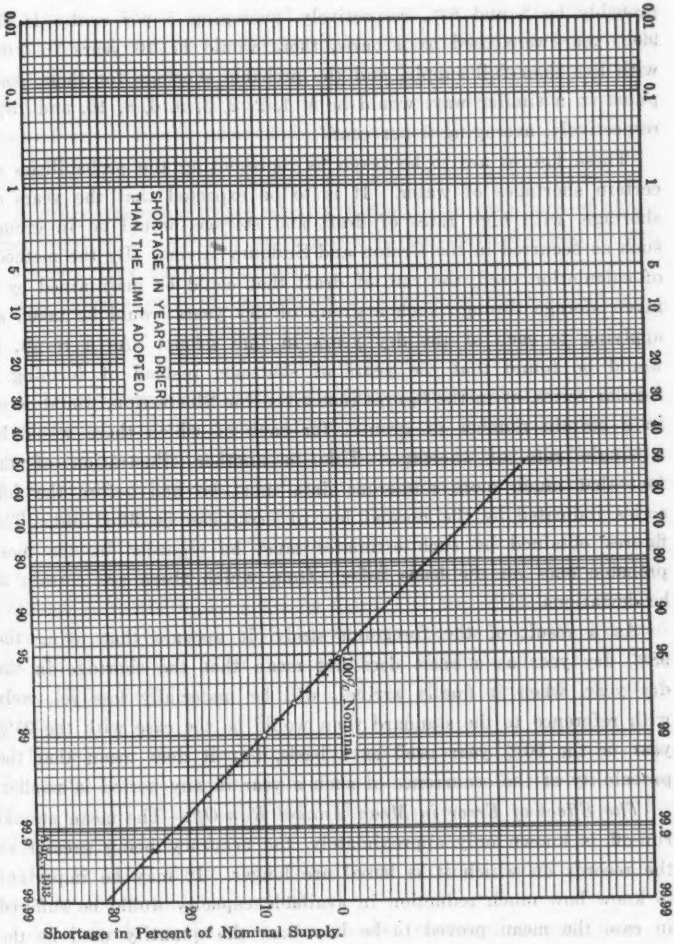

SHORTAGE IN YEARS DRIER THAN THE LIMIT ADOPTED.
100% Nominal
0.01 0.1 1 5 10 20 30 40 50 60 70 80 90 95 99 99.9 99.99
80 20 10 0
Shortage in Percent of Nominal Supply.

Fig. 36.

will be 1, 3, 5, 8, and 14%, respectively, averaging 6 per cent. If the 98% year is taken as a basis, there will be 2 years in a century when the supply will be short, and the shortage in these years will probably be 2 and 8%, respectively, averaging 5 per cent. If the 99% year were used as a basis, there would be 10 years in 1 000 with less than full supply, and the probable shortage for them, computed in a similar way, would be ½, 1, 2, 2, 3, 5, 6, 8, 10, and 15%, respectively, averaging 5 per cent.

These figures are to be taken as representing the probabilities of certain shortages of water. It is to be expected that the years of shortage, with high rates of draft and storage, would be in groups such as occurred in the Croton and Sudbury Rivers. By the methods of calculation used, the rate of draft that could be maintained by a given storage through such a group of dry years would be taken as applying to each of the dry years in that group. As a result, it would be found that the years of shortage, instead of forming a regular series, as in the figures just given for illustration, would occur in a certain number of groups, for each of which there would be a single rate of shortage. This is another illustration of the fact that much more numerous data must be had before the full series indicated by the normal law of error can be filled out. The figures obtained by such estimates must be regarded as the most probable ones on the basis taken, from which there are certain to be deviations.

As a result of the foregoing study, it appears that using the 98% dry year as a basis does not mean that the shortage in the dry year, when it finally arrives, will be materially less relatively with reference to the standard than would be the case with the 95% year or the 90% year used as a basis, but it does mean that the probability of the occurrence of such a year in any period is smaller.

The Effect of Error in Mean Annual Run-Off.—The mean annual run-off is known only approximately, the accuracy being greater as the records on which it is based are longer. It may be important to know how much reduction in available capacity would be suffered in case the mean proved to be less than the quantity used as the basis of the calculation.

With a fixed quantity of storage, the reduction in available draft is not in proportion to the reduction in run-off; the percentage that

is utilized becomes greater as the run-off is smaller. Below the critical storage, with a fixed quantity of storage, 2% reduction in mean annual run-off results in about 1% reduction in the net available supply. Above the critical storage, 5% reduction in the mean annual run-off reduces the net available supply by about 4 per cent. These figures will vary somewhat with different assumed conditions, but they are sufficiently close for use in considering the probable effect of errors in the mean flows that are used.

As to the Accuracy of Averages of Short-Term Records.

A study was made to see whether the probable variations of short-term averages followed the rules given in the textbooks, and if not, to see how they vary from them.

The probable error of the average in a series of random data is:

$$\frac{\text{Probable error in one term}}{\sqrt{\text{Number of terms}}}.$$

It is proposed to see whether the figures representing mean annual flows of a stream for a term of years vary as if each number were drawn out of a bag containing numbers representing all the terms of the series, or whether they occur in cycles in such a way as to modify short-term averages.

To test this point, the relative mean annual flows of the Croton River and Neshaminy, Tohickon and Perkiomen Creeks, were selected. Each has a record of 25 years or more, and the coefficients of variation are near together. All shorter-term records are excluded, and also the Gunpowder River, because of its higher coefficient of variation. The coefficient of variation of the whole series of 120 years is 0.235.

The figures were first divided into sixty series of 2 years each. The sixty average results form a new series, and the probable error of one term in it is ascertained. The same process was followed for other lengths of period.

The probable errors thus found were compared with those computed by the formula just given.

As a check on the work, the number representing each term in the 120-year series was written on a piece of paper, these pieces of paper were mixed and drawn and written down as drawn, forming a new series in which the terms are identical with the terms in the

first series, but in which any tendency to cycles is presumably destroyed. This series was then divided, and the probable errors of short-term averages were determined as had been done in the first place.

The probable errors computed and found in the seven series are given in Table 18.

TABLE 18.

Number of series.	Number of terms in each series.	Probable Error in Average of Each Series.		
		As computed from formula.	Series I, as occurred.	Series II, with the numbers as drawn.
120	1	15.9	15.9	15.9
60	2	11.2	12.4	11.1
40	3	9.2	10.0	9.7
30	4	7.9	10.3	8.5
24	5	7.1	8.0	5.6
20	6	6.5	8.9	5.0
15	8	5.6	7.6	5.0
12	10	5.0	5.7	4.8
10	12	4.6	4.3	4.3
6	20	3.5	5.7	4.3
4	30	2.9	3.0	2.8
3	40	2.5	1.9	2.0

The averages of periods of from 4 to 8 years have probable errors averaging 2% greater than those computed by the formula. In shorter and longer terms the excess is not more than 1 per cent. The same figures as drawn to destroy cycles, if such existed, yielded results differing from the calculated results as often in one direction as in the other. The method of calculation is thus supported, and may be accepted, except so far as it is modified by the existence of cycles in the records. The figures indicate the presence of cycles, and an appreciable, although not large, influence from them in the average accuracy of short-term averages.

Taking this influence into account, to get an average result for which the probable error will not exceed 10% with streams like those used as a basis for this study, a 4-year record is necessary. To reduce the probable error to 5%, a 12-year record is necessary, and to reduce it to 3%, 2%, and 1%, records of 28, 63, and 250 years, respectively, are required.

The probable errors of the means for the eastern streams, for the records used in this study, range from 2.1% for the Hudson to 3.8%

for the Gunpowder. In general, a 3% probable error for these data may be assumed.

By the normal law of error, the average error is 18% greater than the probable error. Using the "probable error" as the standard of comparison, or the measure of magnitude of actual errors, and considering the law of normal error, the following relations may be stated: There is one chance in 2 that the actual error in the mean flow will exceed the probable error; that is, the actual error is as likely to be on one side of the probable error as on the other. There is one chance in 4 that the true mean of a series will be less than the record mean of that series by a quantity greater than the probable error. There is one chance in 11 that it will be less than the record mean by more than twice the probable error; one chance in 46 that it will be less than the record mean by more than three times the probable error; and one chance in 286 that it will be less than the record mean by more than four times the probable error. The chance that the true mean of a series will lie above (be greater than) the record mean by a given quantity is the same as the chance that it will lie below (be less than) by that same quantity, but the chance is twice as great that it will lie either above or below by the given quantity.

Probable Error in the Coefficient of Variation.—The probable error in the coefficient of variation, as determined by short-term records, is also a matter of importance, as the coefficient of variation plays a part only second to that of the mean flow in estimating the storage required for the larger drafts with the methods now proposed.

In computing the coefficient of variation from short-term records, it is better to divide the sum of the squares of the variations by $n - 1$ instead of by n. Otherwise, the values for the coefficient decrease with the shortness of the period. With longer records, the difference is not important. In records as short as from 5 to 10 years, the difference is essential.

The average coefficient of variation for the twenty-four 5-year periods previously described was found to be 0.219 as occurred, and 0.228 with the numbers drawn. These may be compared with 0.235, found for the whole series. The probable error of one 5-year determination was found to be 19.8% as occurred and 23.5% as drawn.

In other words, the probable error in the coefficient of variation deduced from one 5-year term is about 20%, and occasionally the variation will greatly exceed this.

The average coefficients of variation for the twelve 10-year periods were found to be 0.225 for the figures as occurred, and 0.230 for the figures as drawn, and the probable errors in the coefficient deduced from one 10-year series were 12.2 as occurred and 16.0 as drawn.

The tendency of the figures as occurred to run in cycles may be the reason for the lower variation in such figures as compared with the figures as drawn.

The probable error in the coefficient of variation for other periods may be assumed to vary inversely approximately as the square root of the length of the period. The figures for the periods as they occurred are preferred to those for periods as drawn, and on this basis a 17-year record is necessary to obtain a figure with a probable error of 10%, and a 69-year record to determine the coefficient of variation with a probable error of 5 per cent.

The errors in the coefficient of variation in the data deduced for the several streams discussed at this time are thus probably within from 7 to 12% of the truth.

It is clear that short-term records must be used with caution in determining the coefficient of variation, and longer records of other streams, more or less similar to the one under discussion, may often give a better indication of the probable value of this coefficient than short-term records of the stream itself.

An error in the coefficient of variation of the mean annual flow of a stream does not affect the required storage below the critical point. Above this point the effect can be best seen in Fig 33. For example, it appears that, with a given storage, about 5% more water is available when the coefficient of variation is 0.20 than when it is 0.25, all other conditions being equal.

Tabular Statement of Data and of the Probable Errors Therein.—For convenience, the principal data with reference to the several streams are brought together in Table 19. The probable error in the mean flow and the probable error in the coefficient of variation, computed by the methods previously indicated, are shown as percentages of the total. The probable error in the deduction in days' storage, be-

cause of natural storage, is given in days, and is found by the formula:

$$x = \frac{0.674 \text{ standard variation in days}}{\sqrt{\text{number of years in record}}}.$$

The standard variation is taken as the value when about half of the mean flow is utilized.

TABLE 19.—SUMMARY OF DATA.

Stream.	Number of years in record.	Mean annual flow, in millions of gallons per square mile daily.	Coefficient of variation in mean annual flow.	Days to be deducted from normal storage diagram.
Bear Creek.......	11	0.188 ± 9.5%	0.471 ± 12.5%	64 ± 5
Catskill...........	17	1.400	....	5 ± 6
Croton...........	45	1.137 ± 2.4%	0.239 ± 6.2%	23 ± 5
Gunpowder.......	29	0.911 ± 3.8%	0.308 ± 7.7%	91 ± 5
Hudson*........	24	1.135 ± 2.1%	0.156 ± 8.5%	73 ± 4
Manhan..........	14	1.317 ± 3.5%	0.196 ± 11.1%	25 ± 8
Merrimac*........	24	1.020 ± 3.6%	0.259 ± 8.5%	66 ± 4
Neshaminy.......	25	1.083 ± 3.0%	0.226 ± 8.3%	10 ± 6
Pequannock......	20	1.423 ± 3.2%	0.213 ± 9.3%	....
Perkiomen.......	25	1.080 ± 3.0%	0.221 ± 8.3%	33 ± 5
South Fork.......	11	0.053 ± 10.2%	0.505 ± 12.5%	*add* 20 ± 11
South Platte......	8	0.075 ± 9.8%	0.412 ± 14.7%	40 ± 8
Sudbury..........	22	1.085 ± 3.6%	0.253 ± 8.8%	25 ± 5
Tohickon.........	25	1.310 ± 3.4%	0.251 ± 8.3%	2 ± 6
Wachusett........	15	1.136 ± 3.5%	0.202 ± 10.7%	61 ± 5

* No allowance made for water area.

Probable Error in Computed Quantity of Supply, from Short-Term Records.—The errors resulting from errors in mean flow and from errors in coefficient of variation are not cumulative, because these phenomena are closely related, and the existence of a tendency to short-term cycles is an element in the results.

To determine the probable error in results obtained from short-term averages, the figures for each of the 5-year terms and the 10-year terms in the above mentioned series, were used as a basis for estimating the maintainable yield, with a storage equal to 60% of the mean annual flow for the whole term. The relative results would obviously depend somewhat on the quantity of storage assumed. Only one storage figure was used, however, this being selected to represent a reasonably complete development.

In making these estimates, allowances for ground-water and evaporation were omitted, as they would be nearly the same for each series, and in comparative results they are not important. The yield from this storage, computed from the coefficient of variation for the whole series, is 78% of the mean flow. For twenty-four 5-year periods, each

considered by itself, the average computed run-off was 78%, with a probable error of 6.4 per cent. One 5-year period (Croton, 1898-1902) indicated a maintainable yield 22% above the average, and the difference was more than three times the probable error. If this were used as a basis for estimating the yield of the Croton River, the actual yield would be less than calculated by 18 per cent.

The twelve 10-year series give a general average of 78.3% and a probable error for one series of 5.1 per cent. One term (Croton, 1898-1907) indicated a maintainable yield 13% above the mean, the difference being two and one-half times the probable error. The probable errors are shown in Table 20.

TABLE 20.

	In one 5-year term.	In one 10-year term.
Probable error in mean flow	8.0%	5.7%
Probable error in coefficient of variation	19.8%	12.2%
Probable error in available yield, with fixed storage, remaining the same in each series, and equal to 0.6 of the mean flow for the whole term	6.4%	5.1%

As a summary of the entire study, it may be said that the probable error in the calculated maintainable yield of an eastern stream, above the critical point, for conditions like those on the streams investigated, would be a little less than the probable error of the mean flow, and, for 10-year records and greater, may be represented approximately by the formula:

$$\text{Probable error, in percentage} = \frac{0.55 \times \text{coefficient of variation}}{\sqrt{\text{number of years in record}}}.$$

It must be remembered there is one chance in 4 that the true result will be less than the quantity computed in a short-term record by more than the probable error; one chance in 11 that it will be less by more than twice the probable error, and one chance in 46 that it will be less by more than three times the probable error. The Croton records for 1898-1902 may be cited as indicating a deviation from the 45-year mean equal to more than three times the probable error for one 5-year period. Such errors may practically be guarded against in many cases by the fact that a short-term record giving such an unusually high mean, as compared with the true mean, would be certain to be a period in which the rainfall and run-off were so much above the normal that the excess would have attracted attention.

The variations in flow of different areas are such that errors up to the probable error and somewhat above it might easily escape detection in this way; but, with errors two or three times as great as the probable error in one short-term series, the cases would be rare when some local evidence of unusual conditions could not be obtained by careful inquiry that would serve as a warning against placing too much dependence on the short-term average.

Cycles in Run-Off.

On Plate XXXVIII are shown the relative annual yields of the several eastern streams for each year, and also the required quantities of monthly storage to maintain drafts of 400 000, 600 000, and 800 000 gal. per sq. mile of land area from each stream.

This diagram shows that periods of wet weather and dry weather extended generally over the area occupied by these streams, so that in a general way, the fluctuations are somewhat similar.

Wide local differences are shown, and these are much wider in some years than in others. Generally, the fluctuations above the mean are greater in magnitude and shorter in duration than those below the mean. One year, 1889, was notably wetter than any other covered by the records.

Only the Sudbury and Croton records go back to the dry period in the early Eighties, and they do not indicate the recurrence of other years as dry. However, there are some years, notably 1895 and 1900, and 1908, 1909, and 1910, when the required monthly storage on some of the streams was as great, or nearly as great, for some of the drafts, as was the case in 1880 and in 1883 for the Sudbury and Croton.

The general impression obtained from inspection of the diagram is that there are cycles, and that a median line of annual yields will be found to have a high point in 1878, swinging down to a low point in 1882, a summit in 1889, a low point in 1895, a summit in 1903 and another low point in 1910, indicating, in a general way, cycles with periods of from 10 to 12 years, but with some minor summits and low points, which, if taken into account, would shorten the average period.

Long-Term Cycles in Run-Off.—There is no direct experimental evidence sufficient to serve as a basis in investigating the existence and magnitude of long-time cycles. There are some indirect data, especially from rainfall records. The relation between rainfall and

run-off is not exact, but the demonstration of cycles in rainfall records may be taken as evidence of the existence of like cycles in run-off records, although the percentage variation would not be expected to be the same.

Vermeule* gives a summary of long-term rainfall data in the United States, and states:

"A careful study of these long series shows a strong tendency to cycles of high and low precipitation. We are likely to have a period of years of high average rainfall, followed by a low period. We see at once that, under these conditions, differences shown by comparing a series of eight or ten years' length at one station with a fifty or sixty-year series at another, may be entirely differences of time and not of place."

A chart at the same place shows the secular changes in annual rainfall. In the Philadelphia record, in 68 years there are eight well-defined summits, indicating a complete cycle every 8 years. The summits are not of the same magnitude, but indicate a gradual progressive increase up to about 1868, followed by a subsequent decrease.

Arthur T. Safford,† M. Am. Soc. C. E., states:

"It is interesting to note that the average precipitation by 10 year periods rose steadily from 1832 to between 1880 and 1890; since which time the average has begun to decrease. While we should not attach too much significance to this, the fact remains that if we ever have 10 continuous years with as low an average precipitation as fell during the years previous to 1840, a good many water supplies in the state will have to be enlarged.

"These differences in rainfall are so clearly marked that I will show this comparison in the form of the following table and diagram:‡"

"Period Average of	Lowell.	Providence.	New Bedford.	Boston.	Average of 4 Stations.
	Inches.	Inches.	Inches.	Inches.	Inches.
9 Years, 1832-1840.....	37.98	36.87	44.27	41.02	40.03
10 " 1841 1850.....	40.27	41.21	44.94	42.88	42.32
10 " 1851-1860.....	44.14	43.73	44.76	50.09	45.68
10 " 1861-1870.....	45.93	47.56	46.27	57.24	49.25
10 " 1871-1880.....	45.24	47.84	46.88	50.26	47.55
10 " 1881-1890.....	46.47	49.07	48.69	48.48	48.18
10 " 1891-1900.....	44.46	48.89	47.72	45.46	46.63"

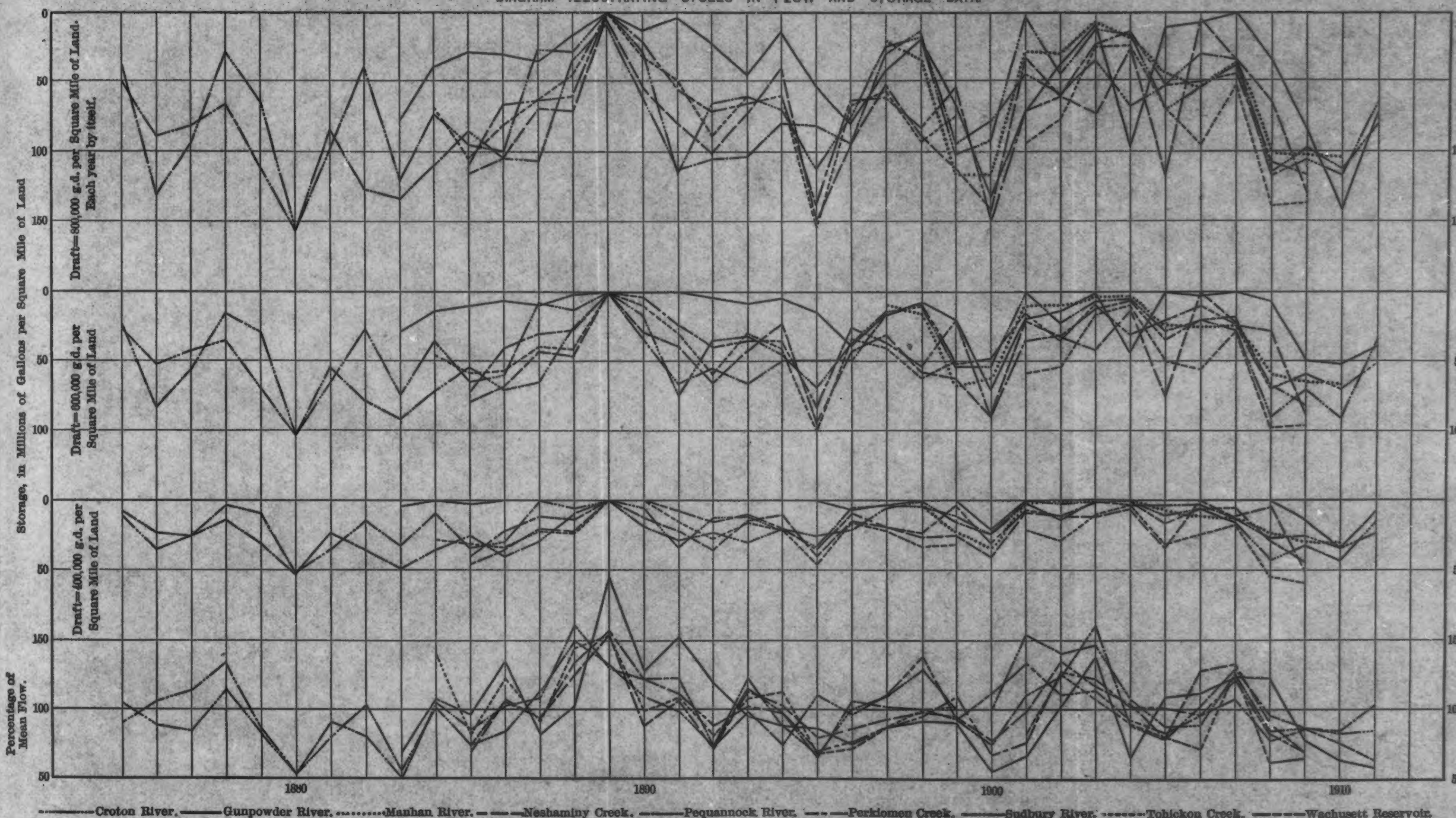
* Geological Survey of New Jersey, Vol. III, 1894, p. 12.

† Report on Fall River Water Supply, July, 1902.

‡ The diagram is not reproduced.

PLATE XXXVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXVII, No. 1308.
HAZEN ON
STORAGE TO BE PROVIDED
IN IMPOUNDING RESERVOIRS.

DIAGRAM ILLUSTRATING CYCLES IN FLOW AND STORAGE DATA.



Turneaure and Russell* state:

"The question of a gradual change in the yearly rainfall is one the solution of which would doubtless require data covering several centuries. The rainfall for a particular locality may average considerably below the mean for many years, after which may follow, perhaps, an equally long period of surplus. In an analysis of several records extending over many years it was found that during an 83-year period at New Bedford, Massachusetts, the averages for 10-year periods were as high as 16 per cent. above the mean and 11 per cent. below; for 60-year periods the extremes were, at St. Louis, 17 per cent. and 13 per cent., and at Cincinnati, 20 per cent. and 17 per cent. For a 25-year period the extreme variations were 10 per cent. for both New Bedford and St. Louis.† From this it is seen that to establish a reliable mean it requires a record extending over a long period of time.

"The variations or cycles above referred to, that extend over several years, are in some cases very marked, but they are very erratic and as yet quite incapable of being predicted."

They also present a diagram in which the fluctuations of averages of precipitation records of certain localities are given. In the curve representing New England there are six clearly marked summits in the period of 60 years, or one every 10 years.

The line showing Ohio Valley stations has eight summits in a period of 60 years, or an average of one every 8 years, and the fluctuations do not correspond in point of time with those of New England. The progressive change in the general average also does not correspond. The total variation is less than in New England where there was a first summit about 4% above the average in 1848. The line then falls to about 4% below the average in 1872, and a second summit 2% above the average was reached in 1885.

A line for the middle Mississippi Valley shows five summits in a period of 35 years, or, on an average, one every 7 years.

Taking all these data together, there is indication of the existence of cycles with average periods of from 7 to 10 years, and with an ordinary variation in rainfall from 10 or 12% above to 8 or 10% below the mean, in the general swing of the curve, and without going to the extremes of 1 or 2 years. There is also indication of a much longer cycle with a range of not more than 4 to 6% above and below the average, and with a period of 40 to 60 years.

* "Public Water Supplies", p. 41.

† "Bulletin D".

Undoubtedly, there may be other longer cycles, with periods running into centuries, but no data are at hand that could be expected to show them if they exist.

As to the Length of Time that a Reservoir will Remain Less than Full.

There is well-grounded objection to the use of a development so large that the reservoir will be less than full through a considerable period of years. When a reservoir remains drawn down for a long time, vegetation grows on its shores, and much additional work must be done, as it refills, to prevent impairment of the quality of the water by the subsequent decay of the submerged vegetation. This may be less important in the future than it has been in the past, owing to the increasing use of methods of treating and purifying water which will result in the removal from it of deleterious substances resulting from such growths and decompositions. However, the matter remains one of considerable importance, and should not be lost sight of in any study for a complete development.

Methods.—The data from which Plate XXXVI was prepared were used and the number of years that the annual flow is less than the corresponding draft and the number of winters during which the reservoir will not refill were counted and the percentages obtained.

From the data referred to above we find for a draft of $1-0.4C$, 27 periods in a total of 252 years, varying in length from 3 to 15 years, during which the reservoir is less than full. These are entered in Column 2 of Table 21 in the order of their occurrence and in Column 3 in their order of magnitude.

If the maximum period of depletion is to be computed for a 100-year period, then the 252 years in the whole series will form 2.52 such periods; and the first two terms in Column 3 and 0.52 of the third term are added to make the sum of 32.7 years for depletion for 2.52 such 100-year series. For one 100-year series the average maximum period of depletion is found by division to be $\frac{32.7}{2.52} = 13$ years. In a similar way, there are 8.4 terms of 30 years each, and the sum of eight first terms and 0.4 of the ninth term is 87.2, and the average maximum term of depletion in each 30-year series, 10.4 years. Similar calculations are made for 20 years and for 10 years.

TABLE 21.

Number of term. (1)	Length of depletion period, in years: $d_1\ d_2 \ldots$ (2)	Figures in Column 2 in order of magnitude: $d_1\ d_2 \ldots$ (3)	Average maximum period of depletion: D. (4)	Years in period for which D is to be found: P. (5)	$\frac{n}{P}$ = number of terms in summation in Column 3. (6)
1	3	15			
2	8	12	$32.7 \div 2.52 = 13.0$	100	2.52
3	3	11			
4	15	10			
5	4	10			
6	5	9			
7	9	9			
8	6	8	$87.2 \div 8.4 = 10.4$	30	8.4
9	4	8			
10	8	7			
11	3	7			
12	9	6	$115.6 \div 12.6 = 9.2$	20	12.6
13	11	6			
14	4	6			
15	3	5			
16	5	5			
17	7	4			
18	3	4			
19	6	4			
20	10	4			
21	7	3			
22	12	3			
23	3	3			
24	10	3			
25	4	3	$165.6 \div 25.2 = 6.6$	10	25.2
26	3	3			
27	6	3			

Similar tables were made for all the other rates of draft represented in Plate XXXVI, and the values obtained plotted, from which smooth curves were drawn, and from these curves values of the average maximum periods of depletion were found, as shown in Table 22.

TABLE 22.—DEPLETION.

Relative rate of draft.	Percentage of years that annual flow is less than the corresponding rate of draft.	Percentage of winters when the reservoir will not refill.	Average maximum period of depletion in one 20-year period.
$1 - 0.4C$	39	58	9 Years.
$1 - 0.6C$	32	43	7 "
$1 - 0.8C$	24	30	5 "
$1 - 1.0C$	17	21	3.5 "
$1 - 1.2C$	10	12	2.2 "
$1 - 1.5C$	3	3	1.5 "

In a 10-year period, the probable maximum period of depletion is about 30% shorter than in a 20-year period; and in a 100-year period, 50% longer.

In using Table 22 it must be borne in mind that periods of depletion as long as those stated will not occur in every 20-year period. The figure given is rather the average length of the longest term of depletion to be expected in each of a considerable number of 20-year periods.

TABLE 23.—RATES OF DRAFT FOR SEVERAL STREAMS WITH THE STATED AVERAGE MAXIMUM PERIOD OF DEPLETION IN EACH 20-YEAR PERIOD.

Stream.	Percentage of water area.	Mean annual flow, in millions of gallons per square mile of land area.	Coefficient of variation.	Years of Continuous Depletion on an Average in Each 20-Year Period.			
				9 years (1-0.4C).	5 years (1-0.8C).	3 years (1-1.1C).	2 years (1-1.3C).
Hudson	....	1.135*	0.156	1.064	0.993	0.940	0.905
Manhan	0	1.317	0.196	1.214	1.111	1.035	0.983
Wachusett	4.16	1.136	0.202	1.044	0.952	0.883	0.836
Pequannock	4.15	1.423	0.213	1.301	1.182	1.090	1.030
Perkiomen	0.00	1.080	0.221	0.985	0.890	0.818	0.770
Neshaminy	0.00	1.083	0.226	0.985	0.887	0.813	0.765
Croton	2.75	1.137	0.239	1.028	0.920	0.838	0.783
Tohickon	0	1.310	0.251	1.178	1.048	0.949	0.882
Sudbury	3.08	1.085	0.253	0.975	0.865	0.782	0.727
Merrimac	2.65	1.020*	0.259	0.914	0.809	0.730	0.677
Gunpowder	0	0.911	0.308	0.800	0.687	0.602	0.546

* Land and water area.

AS TO EVAPORATION FROM WATER SURFACE AND THE ALLOWANCE THEREFOR.

Notwithstanding the historic researches of Mr. FitzGerald, on evaporation, and the data which have accumulated in all the years since his results were published, the actual knowledge of the quantity of evaporation from water surfaces is meager. Primarily, this is because there is no certain way of measuring the evaporation from an actual reservoir. In the best planned experiments, conditions differ rather widely from those of water in an open reservoir. The quantity of experimental data, also, is not great or continuous, and is far from adequate for giving a general idea of the allowances that should be made all over the country.

Mr. FitzGerald's experiments at Chestnut Hill gave a mean annual evaporation of 39.1 in.

At the Lawrence Experiment Station, Massachusetts State Board of Health, H. F. Mills, Hon. M. Am. Soc. C. E., found that the evaporation from water in a wooden tub, 17 ft. in diameter, through a period of 3 years, averaged 29.2 in.

Experiments at Fall River, Mass., reported by Mr. A. T. Safford, in a tank 5 ft. 11 in. in diameter and 3 ft. high, in a coffer-dam, observed for 3 years, with a few winter months missing, indicated a mean annual evaporation of 38.7 in.

At Rochester, N. Y., E. A. Fisher, M. Am. Soc. C. E., found that the evaporation from water in a tub, floated on the surface of the Mount Hope Reservoir from 1891-1909, averaged 34 in. per annum. A similar tub on land, with some results missing, indicated 17 in. per annum more evaporation than from the floating tub. This wide divergence indicates the great difference in results from a slight change in exposure. In summer the water in the tubs was warmer than the surface water in the reservoir by 3 or 4 degrees. It is very likely that the actual evaporation from the reservoir is less than is indicated by the records from the floating tub. When the difference between the conditions in the best arranged exposure and those in an actual reservoir are considered, too much weight must not be attached even to the best experimental results.

Comparison of Rainfall Records with Estimated Amount of Evaporation.—Mr. F. P. Stearns gives the following table:*

"*Table Showing Relation of Evaporation to Rainfall.*

NOTE: + indicates excess of rainfall; — indicates deficiency.

"Month.	Average Year.			Year of Low Rainfall		
	Rainfall.	Evaporation.	Excess or deficiency of rainfall.	Rainfall.	Evaporation.	Excess or deficiency of rainfall.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
January....	4.18	0.98	+ 3.20	2.81	0.98	+ 1.83
February....	4.06	1.01	+ 3.05	3.86	1.01	+ 2.85
March........	4.58	1.45	+ 3.13	1.78	1.45	+ 0.33
April........	3.32	2.39	+ 0.93	1.85	2.39	— 0.54
May........	3.20	3.82	— 0.62	4.18	3.82	+ 0.36
June........	2.99	5.34	— 2.25	2.40	5.34	— 2.94
July........	3.78	6.21	— 2.43	2.68	6.21	— 3.53
August......	4.23	5.97	— 1.74	0.74	5.97	— 5.23
September...	3.23	4.86	— 1.63	1.52	4.86	— 3.34
October.....	4.41	3.47	+ 0.94	5.60	3.47	+ 2.13
November...	4.11	2.24	+ 1.87	1.81	2.24	— 0.43
December...	3.71	1.38	+ 2.33	3.55	1.38	+ 2.17
	45.80	39.12	+ 6.68	32.78	39.12	— 6.34"

*Report, Massachusetts State Board of Health, 1890, p. 345.

Mr. Stearns states:

"It will be seen from the facts presented that the monthly rainfall varies much less during the year than the evaporation; also that in an average year the rainfall is 6.68 inches greater than the evaporation. The average year may be divided into two periods, one extending from May to September, inclusive, in which the evaporation is 8.77 inches greater than the rainfall; and the other extending from October to April, inclusive, in which the rainfall exceeds the evaporation by 15.45 inches.

"In the year of low rainfall the evaporation was 6.34 inches greater than the rainfall. During the warmer months, from April to September, inclusive, the excess of evaporation was 15.22 inches, and during the other six months the rainfall was 8.88 inches in excess of the evaporation. These figures indicate that a pond will not lower by evaporation in a dry summer more than about fifteen inches, even if it receives no water from its water-shed."

The following analysis is presented of the figures used by Messrs. FitzGerald,* Stearns,† and Freeman.‡ The first two publications refer to the Sudbury River, and the data used were the same. Mr. Stearns also used Mr. FitzGerald's data on evaporation as a basis for his calculations. The only differences are in the method of applying the data. Mr. FitzGerald based his calculation on the total area, including water, and Mr. Stearns reduced the figures for flow at the outset to a basis of land area, as is done in this paper. In using Mr. FitzGerald's figures, they have been changed by the necessary calculation to make them comparable with the others. Mr. Freeman's figures relate to the Croton River, and those finally reached are in the shape of a diagram from which the values in the following tables have been scaled, with such calculation as necessary to bring them to the uniform basis. Mr. Freeman also used Mr. FitzGerald's evaporation results as a basis for his allowances.

All the figures in the following tables, therefore, rest on Mr. FitzGerald's work. They are of interest as showing the amount of the corrections actually used, and they are also important because these tables and diagrams have been those that have been used most widely in estimating the capacity of other sources.

The storage, in millions of gallons per square mile of land area, computed for no water area and for 6% water area (6.38% of the land area) are given in Table 24.

* *Transactions*, Am. Soc. C. E., Vol. XXVII, p. 253.

† Report, Massachusetts State Board of Health, 1890, p. 335.

‡ Report upon New York Water Supply, 1900, p. 230.

TABLE 24.

Draft, in gallons per day.	FitzGerald. Sudbury River.			Stearns. Sudbury River.			Freeman. Croton River.		
	No water.	6% water.	Difference.	No water.	6% water.	Difference.	No water.	6% water.	Difference.
100 000........	0.3	6.9	6.6	0.6	8.8	8.2	5	8	3
200 000........	8.8	15.6	6.8	9.4	18.0	8.6	14	20	6
300 000........	28.5	35.1	6.6	29.8	36.1	6.3	29	35	6
400 000........	51.3	56.6	5.3	52.0	57.5	5.5	51	57	6
500 000........	75.8	79.1	3.3	76.5	80.3	3.8	75	82	7
600 000........	100.6	107.0	6.4	102.0	107.1	5.1	100	108	8
700 000........	140.0	157.0	17.0	144.4	161.6	17.2	129	134	5
800 000........	199.1	215.0	15.9	202.3	219.5	17.2	157	162	5
900 000........	334.1	342.0	7.9	346.2	352.2	6.0	207	216	9

The figures under the headings "Difference" indicate the additional storage required to compensate for loss by evaporation with 6% water area. These may be conveniently stated in terms of the depth of water over the area of the water surface. The figures reduced to this basis are given in Table 25.

TABLE 25.

Draft, in gallons per day.	Excess Storage in Depth over Water Area, in Inches.		
	FitzGerald. Sudbury River.	Stearns. Sudbury River.	Freeman. Croton River.
100 000........................	6.0	7.4	2.7
200 000........................	6.1	7.7	5.4
300 000........................	6.0	5.7	5.4
400 000........................	4.8	5.0	5.4
500 000........................	3.0	3.4	6.3
600 000........................	5.7	4.6	7.2
700 000........................	15.3	15.5	4.5
800 000........................	14.3	15.5	4.5
900 000........................	7.1	5.4	8.1
Average.............	7.6	7.8	5.5

All this work has been checked by the writer by an independent calculation, which indicates the substantial accuracy of the methods of calculation which are thus summarized. This was done by making tables of corrected flows for the Croton, Sudbury, and Wachusett records. The actual water area was taken into account for each month, the quantity of rainfall on it and the quantity lost by evaporation, using Mr. FitzGerald's figures for all three sources. In this way the calculated run-off was obtained for an area with no water

exposed to evaporation. By similar procedure, the quantity of run-off was calculated from the assumption that there was 0.1 sq. mile of water area for each square mile of land. The storage required to maintain certain drafts was then computed. These results are shown graphically in Figs. 2, 3, 4, 7, 8, 9, 18, 19, and 20. The excess storage, expressed in inches of depth, for an area having 0.1 sq. mile of water area for 1 sq. mile of land area, as compared with no water area, was found to be practically the same for assumed drafts of 200 000, 400 000, 600 000, and 800 000 gal. per sq. mile. For the Croton, the average excess storage required in years of different degrees of dryness was as follows:

80% Year..................	6.7	Inches.
90% "	7.7	"
95% "	8.6	"
98% "	9.5	"
99% "	10.0	"

The figures are two-thirds of the maximum excess of summer evaporation over rainfall for years of the same degree of dryness. The excess storage required on the Wachusett Reservoir was slightly less than for the Croton, and on the Sudbury slightly more. These differences result from changes in the distribution of the rainfall as much as from variations in the total quantity. It would be easy to introduce refinements in this calculation and extend it, but the accuracy of the data do not warrant it.

Independent Study as to Evaporation.—All the figures thus far given are based indirectly on Mr. FitzGerald's experiments. In the case of three of the streams for which flow data are available, namely, the Croton and Sudbury Rivers and the Wachusett Reservoir, there has been an increase of water area during the period covered by the records. If the excess of evaporation over rainfall on this added water area was large, it would be expected that the records of flow would show the influence of this increase. To investigate this matter, each record was divided into two parts, the latter of which in each case represents a larger percentage of water area. The storages required to maintain flows of 100 000 and 200 000 gal. per sq. mile were taken, and the storage required to maintain the flow in the 95% year found for them. The results are shown in Table 26.

TABLE 26.

	Croton.		Sudbury.		Wachusett.	
	1st series.	2d series.	1st series.	2d series.	1st series.	2d series.
Length of time, in years	24	20	10	12	7	8
Percentage of water area	1.92	3.72	2.56	3.53	2.23	5.85
Storage, in millions of gallons per square mile of land, required to maintain a flow of 0.1 million gallons per square mile of land in 95% year	3.0	3.0	2.0	1.5	0	1.5
Storage, in millions of gallons per square mile of land, required to maintain a flow of 0.2 million gallons per square mile of land in 95% year	10.7	12.1	12.9	8.2	2.4	9.4
Increase in water area	1.80%		0.97%		3.63%	
Increase in storage, 0.1	0.0		−0.5		1.5	
0.2	1.4		−4.7		7.0	
Average	0.7		−2.6		4.2	
Equivalent depth of water over additional exposed area, in inches	2.2		−15.4		6.7	
Weighting factor	44 × 1.80 = 79		22 × 0.97 = 21		15 × 3.63 = 54	

Weighted average for all three, net loss, 1.4 in.

On the basis of the weighted average, a net loss by evaporation of 1.4 in. is shown. The relative loss would obviously be greater in a dryer year. Repeating the calculation for the 98% year, the excess evaporation on a weighted average is found to be 2 in. This is less than one-third of the average quantity deduced in the preceding calculations.

In the Sudbury records the second series clearly represented a less dry time than the earlier series, and a change in this respect was more than enough to offset any influence exerted by evaporation on the increased water surface. In the same way, the second Wachusett series represents a dryer period than the earlier one, and the effect of evaporation is probably less than is indicated by these figures. As far as these data go, they indicate that the allowances made by Messrs. FitzGerald, Stearns, and Freeman were at least sufficient for these streams.

Obviously, the amount of this allowance, which is relatively small in these particular cases, and not a matter of the first importance in calculating the storage on these streams, would become greater with smaller rainfall or with greater relative evaporation such as must be anticipated in many places. On the other hand, for some reservoirs,

even in the dryest years, the rainfall on the water area will exceed the evaporation.

As to the Method of Applying the Allowance for Evaporation.—The whole process may be conveniently divided into three parts. The first consists in excluding the water area from the computation and reckoning the discharge on the land area only. This was done at the outset with most of the data used in this paper.

The second consists in finding the probable mean yield or mean loss from the water area, and adding it to or subtracting it from the estimated run-off from the land area, thereby obtaining a corrected estimate of the mean annual flow from the whole area. It may be noted that the relative variation in yield from water area is greater than in yield from land area. It follows that, as the water area increases, the coefficient of variation of the mean annual flow will increase. With large percentages of water area, this is a matter that must be taken into account. With only such variations as are represented by forming reservoirs for the development of ordinary catchment areas, the change in the coefficient will not be large, and its effect may usually be overlooked.

The third step is to find the amount by which the evaporation will exceed the rainfall during the period of depletion of the reservoir in a dry year. The amount of this excess, in inches over the whole water area of the reservoirs, represents a quantity of water which must be added to the calculated storage, in order to obtain the total required storage to maintain a given draft, or, in the reverse calculation, it must be deducted from the capacity of a reservoir before calculating the maintainable yield from it.

In the case of the Croton, Sudbury, and Wachusett Reservoirs, on the best available evidence, the amount of this allowance should be between 6 and 10 in. This is considerably less than the maximum evaporation in excess of rainfall in all the dry months of a dry year. This latter, from the best data, amounts to about 15 in. The allowance, therefore, is from one-half to two-thirds of the maximum amount that a reservoir without inflow or outflow would lower in a dry year.

The reason the allowance is less than the full amount of such lowering is that the period of depletion due to draft of water at a steady rate does not coincide with the period during which the evaporation is greater than the rainfall.

For other climatic conditions, it is apparent that the allowance would vary. It will increase with the evaporation and will decrease with the rainfall, and it will increase relatively with the length of the period of depletion. That is to say, with a well-marked seasonal fluctuation in rainfall, the correction for evaporation may be greater than with well-distributed rainfall.

For roughly approximate work, the writer suggests that the allowance for evaporation in a 95% dry year may be estimated as equal to the mean annual evaporation, less two-thirds of the mean annual rainfall. Such a rule cannot be regarded as close, and it is probable that additional data will lead to its modification. On present information, the probable error involved by its use will not be very great, except in the case of large and shallow reservoirs.

Storage on a Tributary.

The ordinary condition of storage is in a reservoir on a main stream, so that all the catchment area is tributary to it. It frequently happens that the best reservoir site, or the actual reservoir, is on a branch stream where only part of the catchment area is tributary to it, and the question arises as to how far the storage on such a tributary is equivalent, or nearly equivalent, to storage on the main stream.

This matter was investigated at Springfield in connection with the Borden Brook Reservoir, which, as a first development, had only one-sixth of the Little River catchment area tributary to it. It was also investigated by Mr. R. R. Marsden and the late Richard Hazen, Jun. Am. Soc. C. E., as a graduation thesis at the Thayer School of Civil Engineering, in 1909.

Storage on a tributary is obviously as useful as storage on a main stream, up to the point where the reservoir would be certain to fill in the dryest winters. For streams of the character of those in the East now under discussion, this is represented by about 12 in. of storage for the area back of the reservoir.

Additional investigation, however, has shown that a larger reservoir is filled in the years that precede the dryest year; that it is used in maintaining the flow during the dryest year; and though it may not refill in the winter following the dryest year, that fact is immaterial because enough water will be collected to serve during the year

following the dryest year; and, in the years that follow, the reservoir will be refilled before another extremely dry year occurs.

It was found that with 12 in. of storage on the main stream as the greatest quantity which could be counted on to be refilled with certainty each winter, a reservoir on a tributary holding 18 in. from the area back of it, or 50% more than the maximum size that could be refilled every winter with certainty, was practically as serviceable as an equal volume of storage on the main stream. That is to say, on a given catchment area 1 000 000 000 gal. of storage on a tributary is substantially as useful as 1 000 000 000 gal. of storage on the main stream, up to the point where the area tributary to the reservoir will suffice in the dryest winters to furnish two-thirds of the capacity of the reservoir. With a larger reservoir, there is some further gain with increasing size, but in a diminishing ratio.

How Far Will it Pay to Go in Providing Storage?

In the past the question has been mainly discussed on the basis of providing storage sufficient to maintain the supply through a period of years as dry as those taken as the limiting condition, usually those from 1879 to 1883. We now have data to make an estimate of the probability of the occurrence of years of various degrees of dryness, and it remains to determine which shall be used. Obviously, it is necessary to provide storage sufficient to maintain the supply in as dry years as any that occur with considerable frequency. It is equally obvious that it will not ordinarily pay to provide storage to maintain the full supply through a year so dry that it will recur, say, only once in 100 years. The chances are that a supply now built will have passed through the whole period during which it is able to maintain its intended service, and will have been reinforced with other supplies before the dry year for which it is built would occur.

It has not been infrequent, in American practice, for cities to have somewhat less than a full supply in very dry years. In some cases, where water is being wasted, a moderate shortage may be a blessing in disguise, for it brings about a study of conditions of supply and a stoppage of waste, which is advantageous to the system. This proved to be the case with the recent dry period in the New York City Works. No such advantage results where the services

are all metered and leaks are reduced to a minimum, but, on the other hand, there is inconvenience and loss from a shortage of water. Nevertheless, it is possible, in a very dry year, to cut off some uses of water and reduce the output, and it will be better to do this once in a long term of years than to spend additional money on storage to be but seldom used.

It has been pointed out that, normally, the consumption of water by a city is steadily increasing, and that the supply available at the reservoir fluctuates from year to year with the rainfall. There is only a moderate probability of a very dry year occurring at the same time that the city reaches the dry-weather capacity of the source. In other words, if the supply is designed so that the full service can be maintained 19 years out of 20, there is only a slight probability that the very dry year which cannot maintain the supply will be one of the same years in which the consumption has grown until it has reached the full capacity of the source.

Another consideration that may be taken into account in cases where several systems are near each other is the possibility of buying water from neighboring cities in a very dry year. Growth is always anticipated to a greater or less extent in reservoir supplies, and in other water-works structures, and the chance that all of a group of cities would require the full quantity provided for each at the same time is small. Then there is the chance of getting temporary supplies of inferior quality in times of great emergency. This is more practicable at the present time because of the possibility of disinfecting the water and reducing the danger of infection.

Impounding reservoirs are very stable structures, and subject to but little depreciation. If 5% is the average annual value of capital invested in them by cities, and it costs \$100 per million gallons of capacity to build a reservoir larger, it will cost \$5 per annum for the chance of using each million gallons of extra water when it is needed. If the storage is sufficient to maintain the service for 90% of the years, there would be a chance of selling the first additional water in one year in ten. On this basis, the cost of the water, when needed, would be \$50 per million gallons, or one-half the cost per million gallons of increased reservoir capacity. In other words, if the value of the chance of using the water in the dryest year is worth half the cost per million gallons of building the reservoir larger.

the 90% dry year will logically be used as a basis of calculation. If the value of the chance of using the water in the dryest year is equal to the cost per million gallons of building the reservoir larger, it will pay to build for the 95% dry year, and if the chance of using the water is worth two and one-half times the cost per million gallons of building the reservoir larger it will pay to go to the 98% dry year.

There are matters which practically enter into the consideration of this question and are not capable of definite analysis. The beginning of the dryest year in a long term, which, on the basis mentioned, will result in a considerable shortage of water if the whole supply is then needed, may not be distinguished in its early stages from an ordinary dry year such as recurs at frequent intervals, and does not result in a shortage of water. When the very dry year comes it may be computed that it will be possible to maintain a supply equal to 95% of the normal supply. However, if, during the first half of the period of depletion, water is drawn at the full rate, it will obviously be possible to draw water at only 90% of the full rate during the last half of that period. If the draft was continued at the full rate for three-fourths of the period, the water remaining would only suffice for 80% of the normal output for the remainder. If the extent of the dry period could be foreseen at its beginning, it would be possible to curtail the use of the water at the beginning of the dry period, so that no great hardship would result; but, if the matter of curtailing use is only taken up after most of the water in the reservoir is gone, there may be serious shortage at the end, to avoid which large expenditures would be justified.

A shortage of this kind has a far-reaching effect, and after it there will be a tendency to uneasiness in the beginning of other periods of drought, even when no shortage of water would result.

Such uneasiness may lead to the expenditure of large sums of money for temporary supplies, built in haste and not well adapted to permanent service; and such supplies may not prove to be finally necessary, for the rains will have broken the drought before they are ready for service. Such emergency developments are unfortunate, and it will be worth while to spend a certain additional sum of money to secure increased storage to prevent the chance of needing them, even though the water thus provided may never be required.

On the other hand, it is difficult to persuade the public that a source of supply is being used to the limit of its proper capacity while water runs over the spillway nine years out of ten, and especially when there has been no recent period of water shortage.

The matter is evidently one that must be considered broadly, and an extreme view cannot be maintained. The wisest course seems to be to take a middle basis of estimate, such as the 95% dry year, but to recognize fully its position, and to be prepared at intervals to meet a moderate shortage in supply.

Application to a Specific Case.

The area of the Croton water-shed is 360.4 sq. miles, of which 19.3 are water and 341.1 are land. The estimated run-off from water area is 9.5 in. and from land area 24.0 in. For 19.3 sq. miles of water the production of water is as great as on 7.6 sq. miles of land, and the whole catchment area produces as much water as 348.7 sq. miles of land area. The total storage, including water held by flash-boards, but excluding water in the bottoms of reservoirs not available for supply, is 104 billion gallons. From this an allowance for evaporation equal to 8 in. in depth over 19.3 sq. miles must be deducted, equal to 2.7 billion gallons. The reservoirs, therefore, have a net available storage of 101 300 million gallons, equal to 290 million gallons per square mile of area. A line representing the storage is drawn across Fig. 31, and the values in Table 27 are obtained.

TABLE 27.

Year of what degree of dryness.	Maintainable yield, in thousands of gallons per square mile per day.	Total yield of catchment area, in millions of gallons per day.
80%	1 020	356
90%	970	338
95%	930	324
98%	890	312
99%	869	303

The calculation may be made in another way, using Figs. 33 and 34. The mean average run-off for the Croton River, 349 sq. miles of equivalent land area, at 1 137 000 gal. per sq. mile per day, is 397 million gallons daily, or 1 450 billion gallons per annum. The storage is 104 billion gallons. From this deduct evaporation equal to 8 in. in depth over the water area, 2.7 billions, and add for natural storage

equal to 23 days' supply at an assumed yield of 320 million gallons per day, 7.4 billions, which makes the total net storage, natural and artificial, 108.7 billion gallons, equal to 0.750 of the mean annual run-off.

Referring to Fig. 33 and interpolating between the lines for the coefficient of variation, which is 0.239 for the Croton River, it is found that in a 95% year 82.0% of the mean annual run-off should be utilized, which is equal to 326 million gallons per day. In Fig. 34 it is found that in a 98% year 78.2% of the mean annual flow should be utilized, equal to 310 million gallons per day. These figures check those previously reached, and illustrate a method of computation that would be more frequently used.

It may also be seen, from the position of the point showing the quantity of flow on these diagrams, that the probable maximum time that the reservoir will not entirely fill in a 20-year period is about 5 years, being slightly greater for the larger of the two drafts and slightly less for the smaller one.

These figures mean that, assuming the exact accuracy of the data and methods of computation, with a steady draft of 326 million gallons per day, there will probably be a shortage during some part of one year in twenty. With a like draft of 310 million gallons per day the shortage may be expected in one year in fifty, etc. The figures do not mean that there is only one chance in 50 that the supply will fall below 310 million gallons per day in any given year. They do not mean this because there are other sources of error than those thus far taken into account. Thus there are errors in the measurement of water and in the methods of calculation.

Actual Probability of Shortage in Supply.

If it were required to estimate the quantity of water such that taking all these matters into account there will be only one chance in 20 or 50 that the yield in any given year will fall below it, it ought to be possible to make an approximate estimate of such amounts.

The following is suggested as the general means of procedure, and is presented as an illustration and not with a view to attach importance to the particular figures used. The foregoing figures are plotted on probability paper on Fig. 37, and the line extended crosses the 50% line at 390 million gallons per day, and this will be taken as

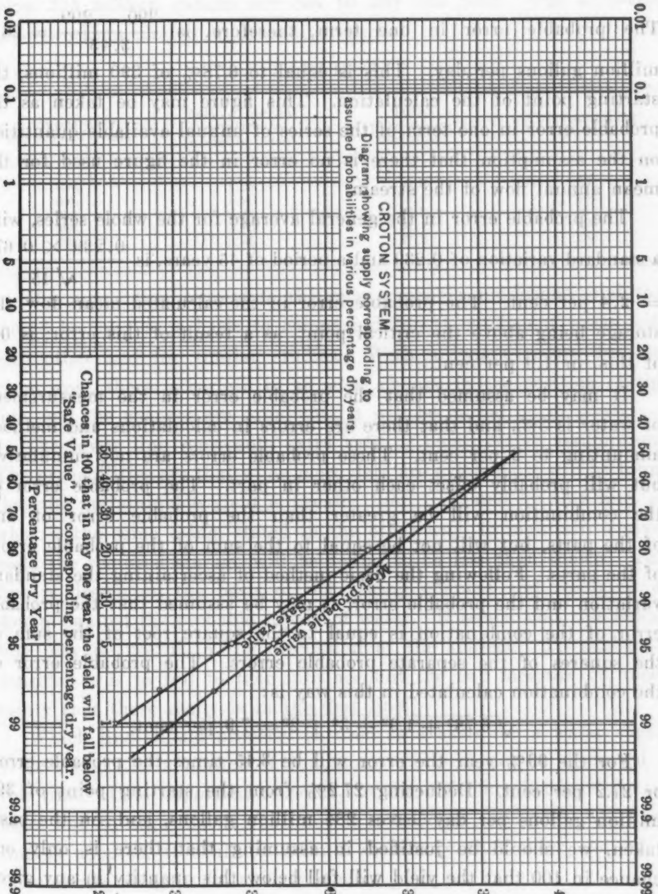

CROTON SYSTEM.
Diagram showing supply corresponding to assumed probabilities in various percentage dry years.
Most probable value
Safe value
Chances in 100 that in any one year the yield will fall below "Safe Value" for corresponding percentage dry year.
Percentage Dry Year
0.01 0.1 1 5 10 20 30 40 50 60 70 80 90 95 99 99.9 99.99
50 40 30 20 10 5 1
280 300 320 340 360 380 400
Yield, in Millions of Gallons Daily per Square Mile

FIG. 87.

the starting point for the calculation. It crosses the 99% line at 299 million gallons per day. The difference between the 50% and the 99% year by the normal law of error is 3.45 times the probable error. The probable error in one term, therefore, is $\frac{390 - 299}{3.45} = 26.4$ million gallons per day. This is equal to 6.78% of 390 millions, the starting point of the calculation. This figure may be taken as the probable error in one term of the series of annual available quantities, on the assumption that there is no error in the figure used for the mean annual flow of the stream.

The probable error in the general average for the whole series, with a standard variation of 0.239 and a period of 45 years, is $\frac{0.239 \times 0.674}{\sqrt{45}}$ $= 2.4$ per cent. The probable error in the calculated mean flow, the storage being above the critical point, as a result of this error, is 0.8 of this, or 1.9 per cent.

It may be assumed that the probable error in the measurement of water is 3%, and that there are errors in calculations and methods amounting to 2 per cent. These probable errors are not cumulative, but will probably offset each other in part. The probable error of the combination will be greater than the probable error of any of the parts, but will not be equal to the sum of the probable errors of the parts. Following the basic method of ascertaining the standard variation and the probable error, it may be assumed that the probable error of the combination is equal to the square root of the sum of the squares of the separate probable errors. The probable error of the combination calculated in this way is:

$$\sqrt{6.78^2 + 1.9^2 + 3^2 + 2^2} = 7.9 \text{ per cent.}$$

For the 99% year the error will be 3.45 times the probable error, or 27.2 per cent. Deducting 27.2% from the starting point of 390 million gallons per day leaves 284 million gallons, and, on the basis taken, we should be justified in assuming that there is only one chance in 100 that the yield will fall below this quantity in any given year. Other values are given in Table 28.

Taking the 95% year as a basis, there is a 5% chance that there will be a shortage in flow in any given year with a steady use of 326 million gallons per day, this being based on the assumed accuracy of all the data and the general average of the whole term. If, in addi-

tion, insurance is to be written against the errors in the general average, in measurements and calculations, to the extent just indicated, then there would be a 5% chance that the yield would fall below 315 million gallons per day in any one year. Fig. 37 shows these two lines plotted on probability paper.

TABLE 28.

Comparative dryness of year.	Ratio of actual variation to probable error.	Total percentage variation.	Corresponding yield.
80%	1.25	9.9	351
90%	1.90	15.0	331
95%	2.44	19.3	315
98%	3.05	24.1	296
99%	3.45	27.2	284

The upper line represents the most probable conditions, and this should be used as a basis to compare different catchment areas with each other and for similar studies. With a longer record for any given area, there is certain to be some change in the position of this line, and the chances are equal that it will be moved up or down.

The lower line represents extra safe or conservative assumptions. The probabilities are that when more data are available it will be found that the maintainable yield is greater than indicated by this line, or, what is the same thing, stated in the other way, to maintain a given yield the required storage will be found to be less than shown. This line is drawn on the basis of not taking chances and resolving all doubts against the probable supply of water. It seems that this second line may be useful in the discussion of the subject. To neglect to consider it is to fail to take into account the full measure of uncertainty that there really is in the first line.

Which line should properly be used as a basis for a water supply project must be a matter of judgment, and will not be discussed at this time. It is important to point out the difference between the two bases and to urge that it shall not be overlooked; and that, whichever line is adopted, it shall be defined so clearly that there can be no misunderstanding as to what is meant.

CONCLUSIONS.

The study of storage and maintainable flow of streams involves two classes of relations: First, those which are more or less definite and fixed, and can be analyzed by definite processes; second, those

which rest on conditions too complex to be understood and analyzed by ordinary means of procedure.

The variations of the second class follow, in a general way, the normal law of error, although some well-defined deviations from it have been found.

This paper presents a graphical method of reaching an approximate solution of the problem in probabilities presented by these variations, and permits definite results to be obtained from the data of stream flow and storage. These results are definite, although not exact. They may be used with confidence within limits of probable error, and these limits can be determined.

Arranging the data in this way permits results to be obtained for years of specified degrees of dryness, and thus it is possible to make studies on a more definite basis of the first above defined class of relations. It is found, also, that if the storage required to maintain a flow at all times in an average year equal to 50% of the mean flow of the stream is determined, and if this storage is expressed in terms of days' supply as the assumed rate of draft, then the increased number of days' storage for higher rates of draft are approximately constant for different streams; and, on the other hand, the reduction in the number of days' storage, with equal reductions in the proportionate rates of draft, are nearly the same for all streams.

In a similar manner, the increase in the days' storage, in a year so dry that only one year in twenty is dryer than it, is greater than the storage required in an average year by quantities which do not differ widely for different streams. In this way it is possible to form normal inclusive curves of storage for various rates of draft and for years of varying degrees of dryness that will apply to several streams. The actual storage required for any stream will usually be less than the normal by an amount which is approximately constant for various rates of draft and for various degrees of dryness. In this way it is possible to find how the storage required for a particular stream compares with the normal, and afterward to form a judgment of the storage required for other rates of draft and for years of varying degrees of dryness. Thus an estimate may be made which will be more accurate than could be obtained from the records of one stream, however long continued.

It has long been known that in some streams the flow is steady and in others it is variable. This fact is taken into account in this study, and a figure is used as an index of the degree of this variation. This figure, called the Coefficient of Variation, can be obtained from the records of flow of any stream covering a sufficient period, and, when obtained, is used as a basis for comparing the data for that stream, and estimating the probable storage and other matters in connection with relatively high rates of draft. By the use of this Coefficient of Variation it has been found possible to get an approximate expression for the storage required to carry the surplus water of wet years over to dry years, and to put this in such general terms that it applies almost equally well to eastern streams having the least variation in their flows, and to some western streams for which data were examined having many times greater relative variations in their flows.

The records of any one stream, even of those for which the longest records are available, are too short to establish with accuracy the probabilities of the occurrence of very dry years. The records of all the streams used in this study combined into a single series, afford a better basis for estimating the probabilities of the occurrence of very dry years than the records of any one stream. The writer believes that the basis deduced from the records of all these streams is a safer one to use than the records of any stream now having an available record, even when applied to that particular stream. This is especially true where relatively high storages and rates of draft are considered. In other words, the normal storage diagrams, as applied with suitable allowances for local conditions, are believed to afford a more reliable basis for estimating the probable yield of a stream than can be obtained by any method now available from the records of that stream alone.

The methods herein described, therefore, should afford, not only a means of interpreting more accurately the data available for streams that have been gauged for long periods, but should make it possible to make better estimates of the probable yields of streams which have not been gauged, or have been gauged only for short periods, and the performance of which must be judged, for the most part, from records of other streams similarly situated.

The precision of results reached may be disappointing to those who have made calculations of run-off and storage carried to several

decimal places. Such a degree of precision is not to be inferred from any run-off records at our disposal.

The use of the methods herein proposed makes it possible to estimate the probable errors in the results reached; and frank recognition of the large probable errors in many of the results cannot fail to be advantageous.

The methods of analysis herein proposed seem to be capable of application to other engineering problems, and their use will lead to more accurate knowledge of phenomena which contain large and unexplained elements of variation.

DISCUSSION

HIRAM F. MILLS, HON. M. AM. SOC. C. E. (by letter).—The writer is unable at present to make a critical study of this subject, but would like to ask Mr. Hazen if he has sufficiently provided for the condition —observed many times in New England rivers—that a very small run-off is often followed by a second year of small rainfall, which gives more trouble because following after ponds and ground-water were lowered? Mr. Mills.

Exceptionally, we have the third very dry year, as in 1911, when storage reservoirs were generally empty, and very great anxiety was relieved by a moderate rain in October and more in November. This third year, of course, he would regard as the exceptional year, when emergency supplies or borrowing may have to be resorted to; but the writer is not quite sure that the second very dry year—which often occurs—is provided for.

T. U. TAYLOR, M. AM. SOC. C. E.—Among the elements or factors entering into the problem of the storage of water for municipal water supply, the author mentions the following: Mr. Taylor.

1.—The size of catchment area or water-shed;
2.—The mean annual run-off per square mile;
3.—The portion of water area and the loss by evaporation from it;
4.—The natural storage in lakes, or in deposits of sands, gravels, or pervious materials;
5.—The regularity of annual flow.

Having spent more than half a century in the West, and having been more than half that time a close student and observer of water conditions in the great southwest section of the United States, the speaker welcomes any discussion of storage problems, even though the data for the most part refer to sections of the country with conditions unlike those of his own. Such discussions are valuable, although they may serve the sole purpose of convincing engineers that the data for the East cannot be applied to the West, and that the man who does this brings disaster on the enterprise and should bring it on himself. However, such discussions are valuable, whether for municipalities, irrigation, or power. In the Southwest, especially in Texas, all these problems have to be considered, and it sometimes happens that two, or perhaps all three, storage problems may occur on the same river.

Two of the elements entering into the problem, as mentioned by Mr. Hazen, can be omitted in Western or Southwestern problems,

Mr. Taylor.

namely, the water area in the water-shed and the natural storage in lakes. The three fundamental factors in Texas are:

1.—Area of water-shed;
2.—Regularity (or irregularity) of flow;
3.—Run-off.

The area of the water-shed can be ascertained with a reasonable degree of accuracy, and the regularity of flow can, in general terms, be ascertained by local testimony—and it is about the only factor in the problem where the testimony of the oldest inhabitant is worthy of serious consideration. For accurate calculations, however, it would be necessary to have hydrographs constructed from daily records of flow, and these can only be obtained by gauging stations established and maintained with unbroken records of gauge readings and of annual rating curves. The third factor, the run-off, is the one which contains possibilities of the gravest error. Any attempt to apply run-off data obtained in one section of the country to a locality or stream in another section is likely to result in great damage, unless Providence is kind. It is full of the utmost danger, as many wrecks in the West will bear silent and bitter testimony. The regularity of flow, the quantity of daily flow, the mean monthly flow, the mean annual flow, and the run-off are data that can be obtained only by careful observations and measurements for the locality, station, and stream concerned. The run-off for the same stream, at different stations along its course will be found to vary greatly. The speaker calls to mind a case where the run-off on a stream at one point is 0.1 sec-ft. per sq. mile of water-shed, and at a point 1 mile above it is only 0.01 sec-ft. A stream may course its way through a coastal plain of a reasonably flat country, then enter a canyon section where floods occur with torrential violence, and then again, in its upper reaches, it may enter into a flat upland section. It will be found, and, in fact, has been found, that the run-off and flow in second-feet per square mile of water-shed differ greatly. The problems in the West are largely concerned with the rainfall, run-off, mean flow, and the necessary storage capacity to equalize the flow or to equalize it sufficiently for the municipality, irrigation project, or power problem.

Careful daily records and observations on the Colorado River of Texas, at Austin, Tex., for the last 14 years, have shown that the average or mean flow for this period has been 1 820 sec-ft. The water-shed of 37 000 sq. miles would give a mean flow of $\frac{1}{20}$ sec-ft. per sq. mile. The lowest mean annual flow for this period was $\frac{1}{74}$ sec-ft. per sq. mile, and the largest flow on record, occurring at the time the Austin Dam failed, April 7th, 1900, amounted to less than 4 sec-ft. per sq. mile. The lowest flow on record occurred from August 15th to 20th, 1910, when the run-off was about $\frac{1}{1800}$ sec-ft. per sq. mile.

The mean annual rainfall on the water-shed of the Colorado above Austin for the period was practically 24 in., but the mean flow for the period was only 1 820 sec-ft., which is equivalent to a run-off of 2.88%, or 0.7 in. Mr. Taylor.

With these staggering facts available, a $4 000 000 enterprise was established in a neighboring territory, when, after unusual rains, the engineers awoke to the fact that, in using an assumed run-off derived for other regions, they had omitted the most vital factor, and had, according to their own admissions, over-estimated the possibilities of the scheme by a large percentage. The money had been spent, the water-shed was there, and the rains came, but the run-off refused to run with the vigor that it was assumed to display on such occasions. The only plea of defense is that the engineers were misled, as to the run-off, by a book on irrigation, when the book made no statement as to the run-off in the territory mentioned.

A large municipality authorized the construction of an impounding reservoir and its appurtenances at a cost of $1 000 000, on an assumption of a run-off of 19%, and again there was a failure to come up to expectations.

The run-off is a complex factor, depending on topography, vegetation, kinds of soil (whether cultivated or uncultivated), rainfall, distribution of rainfall as to time of year and as to growth of vegetation, and, what is still more vital, the condition of the soil at the time of the rains. Personal observation has convinced the speaker that a 2-in. rainfall in 24 hours may at one time give a run-off of 25% and at another time no run-off whatever. In 1910 there was the lowest flow of the Colorado River at Austin, but it was not the year of least rainfall, nor the year of lowest mean annual flow. Each stream is a problem unto itself. An experienced engineer will go very slowly in such matters. Like Davy Crockett, he will be sure he is right, and then go ahead, and not, like many, "rush in where angels fear to tread."

The Colorado of Texas has a canyon section or power section, a municipal section, and an irrigation section. At present, due to a lack of storage, neither the power section nor the irrigation section is reliable as a revenue producer, as the flow is spasmodic and irregular. The construction of storage reservoirs on this river is a simple and entirely feasible problem, and it would be an easy matter to store about 20 000 000 000 cu. ft.; and this, in the dryest years, would give a minimum flow of 800 sec-ft., which, with the total head of 240 ft. that could be obtained by the various dams, would give about 20 000 h.p. Then, in addition, this reliable and equalized flow would insure the irrigated rice crop 250 miles below, near the coast. Altogether, the Colorado offers one of the best problems for storage known to the

Mr. Taylor. speaker. Its records as to flow, area of water-shed, run-off, dam sites, and the demand for power and for reliable flow for irrigation, have all been worked out, and there need be no assumptions as to any factor.

The method suggested by Mr. Hazen for rating the dry years is safe, and would fit any section of the country, because each river would be writing its own biography. It might sound a little more comforting to the people of the West if the percentages were given in "wetness" instead of "dryness". The dryness feature is obtruded on them with sufficient emphasis already, because it is a land

"Where the prairie dog kneels,
On the back of his heels,
And silently prays for rain."

Mr. Marsh. F. B. MARSH, ASSOC. M. AM. SOC. C. E. (by letter).—This paper bids fair to be the starting point for much future consideration of this hitherto intangible question of the proper storage to be provided in municipal impounding reservoirs. For this reason the writer wishes to call attention to a factor which, although of relatively minor importance, yet becomes worthy of consideration when a water-shed has been developed well toward the economic limit, as, for example, the Croton.

In allowing for evaporation in a well-developed water-shed, the area of water surface should not strictly be taken as that with all the storage full, because such a condition obtains generally for only a few weeks during the year, and in many years not at all. Moreover, the tendency is for it to occur at a season when the evaporation from water surfaces is small. The maximum evaporation occurs during the summer, when, as a rule, the storage has been depleted and the water surface is, therefore, smaller.

Opinions will probably differ as to what should be taken as an average condition of the reservoirs, but for present Croton conditions, at least, it is probable that the water surface corresponding to a condition of reservoirs two-thirds full would be a fair assumption. Certainly it should not be less than one-half.

The fact that the area of water surface is a continually variable quantity has led the writer to prefer stating catchment areas in terms of the total area, land plus water, rather than land area only, as used by the author. The correction for evaporation can apparently be made as readily on one basis as the other, and it has always seemed to the writer more convenient and logical to use the gross area, and less likely to confuse those who may have occasion to make practical use of the figures.

The author's warning that actual records of an individual stream, taken without reference to other data, are likely to be seriously misleading, even if they cover a considerable period, is strongly exemplified by the Croton record, where the average flow for the 18 years from 1869 to 1886 was 346 000 000 gal. daily, while for the next 18 years, 1887 to 1904, it was 449 000 000 gal. daily. Mr. Marsh.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The writer is much pleased with the results of the investigation given by the author. In May, 1901, he wrote a paper on this interesting subject for the American Water Works Association. This paper was intended only as a rough general guide to aid the judgment of engineers who were in trouble. Since that time he has been called on repeatedly to explain. Accordingly, he gives the following specific case: Mr. Le Conte.

Let x = average rainfall for the 3 dryest years on record.

The average run-off $= \dfrac{x}{2}$

The average catch $= \dfrac{x}{2} \times \dfrac{1}{4} = \dfrac{x}{8}$.

Assume x $= 40$ in.

Then average catch $= \dfrac{40}{8} = 5$ in. per annum.

$5 \times 17\,400\,000$ gal. $= 87\,000\,000$ gal. per sq. mile; continued for 3 years $= 260\,000\,000$ gal. per sq. mile.

Where local experience indicates that it takes 2.5 sq. miles of water-shed to furnish 1 000 000 gal. per day in a long dry spell, then $260 \times 2.5 = 650\,000\,000$ gal. storage, which is necessary to insure a supply of 1 000 000 gal. per day in a long dry spell.

Referring to Fig. 32, the writer is pleased to note a practical corroboration of this result. The Croton storage is given as 101 300 000 000 gal., and the water-shed as 360 sq. miles. This makes a storage of 290 000 000 gal. per sq. mile, for New York. The Spring Valley Reservoirs, at San Francisco, Cal., have 30 000 000 000 gal. storage for 30 sq. miles of water-shed, or, 1 000 000 000 gal. storage per sq. mile.

Then, $\dfrac{1\,000}{290} = 3.45$ times as much storage as is required in New York. Why? Simply because at San Francisco it takes 5 sq. miles of water-shed to furnish 1 000 000 gal. per day, whereas in New York it only requires 1.5 sq. miles to deliver that quantity. In the New York case then, $260 \times 1.5 = 390\,000\,000$ gal. storage, and in the San Francisco case $260 \times 5. = 1\,300\,000\,000$ gal. storage.

E. P. Goodrich, M. Am. Soc. C. E. (by letter).—What Mr. Hazen has done in the design of probability paper is unique, and, with easily obtainable information as to the meaning of lines drawn on it in re- Mr. Goodrich.

Mr. Goodrich. lation to probability formulas, his new device should be of almost as much value to engineers as the well-known logarithmic paper.

The writer believes that the work Mr. Hazen has done in applying the theory of probabilities to the data on annual stream flow and required storage, is a material advance in engineering science. Mr. Hazen has shown conclusively, in the writer's opinion, that the methods of computation to be followed in utilizing the theories of probability are applicable to water problems. The next step in the solution must be what is normally called that of inverse probability or the breaking up of the probability curve found from actual data in any case (such as that prepared by Mr. Hazen and shown in Fig. 29) into several possible component probability curves. This work is exactly comparable with the analysis which is now so common with reference to tidal and similar observations, which have been found to follow compound harmonic relations, breaking up this combination curve into the several simple harmonic ones which, when recombined, will produce the actual result observed in Nature. In the case of the harmonic curves, the tracing back of the several simple harmonic functions to their causes has been found relatively simple. It is similarly to be expected that, if Mr. Hazen's combination probability curve can be analyzed into two or more simple normal probability curves, in combination with one or more other simpler curves, the causes operating to produce the latter can be discovered, and further refinements can be worked out in the engineering of water-works design.

Several explanations of the skew curve, which Mr. Hazen found with regard to run-off data, are possible. One such explanation, which seems to the writer more than possible is that a normal probability variation is superposed upon a condition which would produce a periodic variation of long range. Evidently, if the data happened to have been secured for years which range near the minimum of a periodic fluctuation, the exact conditions, with reference to the data found by Mr. Hazen, would be reproduced. In the paper Mr. Hazen makes reference to such periodic variations in rainfall and run-off, and it is the writer's idea that the skew nature of the curve found by him indicates, primarily, that the data do not cover a large enough range of years to be entirely conclusive in the deductions which can be drawn. In the meantime Mr. Hazen's work will represent the acme to date.

Mr. Tighe. JAMES L. TIGHE, M. AM. SOC. C. E. (by letter).—The ingenious and unique methods of analysis followed in the production of this remarkable paper, and the conclusions and results therein set forth, must be of the greatest interest to all hydraulic engineers.

In the earlier days of providing domestic water supplies from catchment areas, little attention was given to the relation between run-off and storage, with the result, naturally, that deficiencies gen-

erally occurred in the dryer years. After a time these deficiencies led to the establishment of empirical rules or formulas for storage based on low rainfalls, or, where run-off records were available, to the tentative and laborious methods of tabulation. Mr. Tighe.

It was not until the early Eighties that W. Rippl, Docent at the Royal Technical High School at Gratz (Styria), made public his graphical method for the calculation of storage known as the mass-curve method. As originally given, this method was followed in such calculations until the report on the water supply of New York by John R. Freeman, M. Am. Soc. C. E., was published in 1900. In this report a more convenient mass-curve method was shown, in which a single curve served for all rates of draft, whereas, in the original Rippl method, a new curve had to be made for each rate.

In this mass-curve method and in the other methods for calculating storage, it was generally sought to calculate from the rainfall or stream-flow records of the particular catchment area in question, or, if these were not available, from the rainfall or stream-flow records of some other catchment area applicable, the storage required to maintain certain drafts through the dryest period covered by the records, and little if any attention was given to other important phases of the question, outside of evaporation, until the presentation of Mr. Hazen's paper.

In this exhaustive analysis, some new ideas relative to storage, which were never brought into the question before, have been treated at great length. Of these, the amplification of the dry years and their recurrences, the normal storage diagram, and the coefficient of variation may be mentioned. For the latter, hydraulics is under great obligations to the author, inasmuch as it must be admitted that, up to the present time, the science gave no yard-stick whatever by which the variation in stream flow could be determined.

The coefficient of variation proposed is an ingenious one, and will not only serve the purpose for which it is used in the paper, that is, for storage calculation, but will enable a comparison to be made of the variation of streams, not only in one section or territory, but all over the world.

As the variation of a stream, however, is influenced, among other things, by the area of its catchment, inasmuch as the greater this area, the less variation the stream is likely to have, perhaps the author at some future time could expand this idea to cover the catchment area. This might not be included in terms of the coefficient, but possibly in some other way so that when comparing two or any number of streams, relative to variation, their catchment areas would be brought into the comparison.

In regard to the normal storage diagram developed, this, if the writer understands it correctly, is not based on the actual records of

Mr. Tighe. the flow of any one particular stream, but on an artificial record formed from the records of the flow of all the streams used in the analysis. Such an artificial record makes a series covering a period of 300 years, or thereabouts, and may be open to the objection suggested by the author, that it does not reflect any climatic changes which might occur in the actual series, were it not for the fact that such changes or movements are on such a large scale of time that any objection of this kind may be disregarded.

The utility of the normal storage diagram is shown by the fact that by this method the storage required on any stream, to maintain a certain draft, can be determined more accurately than from the actual run-off records of the stream itself, unless perhaps these records covered an exceedingly long term of years, many times longer than the longest now in existence. This diagram method is a great advancement in storage calculation, and very likely will be used as the standard method in such calculations in the future. Even were it the case that actual run-off records could be depended on for storage calculation, this normal storage diagram would be valuable as a check.

A very important point stated by the author is that the storage on a tributary, within certain limits is as useful as that on the main stream. This no doubt is true within the limits stated in respect to reasonably large storage, but is it true in respect to small storage where the capacity of the reservoir and the draft thereon would be such that the reservoir would empty and fill several times a year or several times in the summer season? For instance, if the capacity of the Borden Brook Reservoir, of the City of Springfield, referred to by Mr. Hazen, was only 70 000 000 gal., would this reservoir, with its 8 sq. miles of catchment, be as efficient as the intake reservoir of the same capacity with its 48 sq. miles of catchment, assuming that the pipe line had a discharging capacity of 20 000 000 gal. per day?

The writer has made no test figures on this, yet it seems to him that the intake reservoir would be the more efficient, as it would be possible for it to fill more often in the summer season, because of its larger catchment, than the Borden Brook Reservoir.

Another point that should not be overlooked, relative to storage on tributaries, is the water that is lost by percolation and evaporation in its course from the storage to the main stream and intake, especially if conveyed intermittently in the old stream bed.

Although considerable attention has been given to the loss of water by evaporation from the surface of reservoirs, it seems that much less has been given to the loss of water by percolation from reservoirs.

When a reservoir is built and filled, the level of the ground-water above the dam is raised and, consequently, some extra storage capacity is provided beyond the apparent capacity of the reservoir, especially if the ground surrounding the reservoir is porous.

Mr. Tighe.

On the other hand, because of the ground-water being raised above the dam, if there is any connection between this and the ground-water below the dam, the flow of the latter, owing to the higher head, must be increased, and this increase will come or be fed from the reservoir. It can be argued, of course, that, with the foundations of a dam resting in impermeable material, percolation from the reservoir, under the dam at least, might be considered negligible. The writer would like to get the views of the author and of others on this matter, as he thinks that the loss of water from the average reservoir through percolation is more of a factor than is supposed. If this is the case, it seems that, in storage calculation, especially as other phases of the question are being so closely analyzed, percolation should also be considered.

In conclusion, the writer, under whose supervision the daily flow of the Manhan River, furnished the author, was measured, and who naturally has been interested in and has given some attention to the matter of stream flow and storage, considers this paper the most comprehensive analysis that has ever been presented on the subject. The author certainly deserves the thanks of all hydraulic engineers for the exhaustive manner in which he has treated the subject and the new light which he has thrown on it, especially those phases of it which hitherto have been neglected.

Mr. Cory.

H. T. CORY, M. AM. SOC. C. E. (by letter).—Occasionally in life, one "comes to scoff and remains to pray." That, metaphorically speaking, is the writer's experience with Mr. Hazen's paper. In general, water storage required depends primarily on the run-off (including quantity and distribution throughout the year), and its estimation is a very dangerous and complex proceeding. The writer has seen such disastrous results from attempts to apply run-off data and run-off relationships from precipitation obtained in one section of the country to a locality or stream in another section, often near-by, that Mr. Hazen's paper was taken up in a spirit of impatience. So many tragic wrecks are strewn throughout the Far West from this sort of thing, that it is difficult to be tolerant of the elaborate methods constantly being suggested by ingenious mathematically inclined engineers for "deducing" discharges of ungauged streams. The instances cited by Mr. Taylor are very pertinent, and the writer knows that the particular cases he mentions are presented fairly in his discussion.

It was largely a matter of luck, therefore, that the writer was led to glance over Mr. Hazen's paper in sufficient detail, before throwing it aside, to discover that it was not another and probably the most serious proposal yet made to apply mathematical methods to insufficient data in an endeavor to use observed run-off data covering a short period

Mr. Cory. of time, with a view to "expanding" them into records of sufficient length apparently to justify the design of hydro-electric engineering works to the third decimal place.

The paper, as a matter of fact, is of a totally different character. It was read with very great interest and profit—also with very considerable difficulty, in spite of the fact that the writer, as a student, was given the method of least squares more fully than is usual in American technical institutions, and, in addition, taught it and its applications in engineering work to his students for several years while in charge of engineering departments in two of the larger American universities. It is probable that most members of the Society found it still more difficult to follow the argument.

These two things are probably responsible for the little comment really pertinent to the paper itself which has been brought out. This small amount of discussion is greatly to be regretted, because the contribution, as a matter of fact, is most noteworthy, in several ways. The method of examining hydrographic data itself is of striking importance and interest, as well as the application of graphical methods to the theory of probability, or method of least squares, as it is more usually called.

The title and the context show that the author considers only storage for municipal supplies, and limits his conclusions to regions where hydrographic conditions are similar to those obtaining in the North Atlantic States. The method of analysis, however, is a general one, and the writer will consider its application to general hydraulic work in the Far West, where irrigation and power development overshadow municipal uses. From such a standpoint, some statements and methods of treatment, although quite justified in the paper as presented, require some modifications. The discussion which follows has to do very largely with such features, and, as a whole, is from that standpoint exclusively.

It is unfortunate that the first paragraphs of the paper were presented in just the form they were, because, at the very outset, they give the reader—at least they gave the writer—an erroneous conception of its scope.

"There is undoubtedly a definite relation between the storage provided in an impounding reservoir on any stream and the quantity of water which can be supplied continuously by it. The relation, however, is a complex one, and our knowledge of its character is limited. The following study is made to see how far it may be possible to separate this complex relation into parts, some of them being of such a nature that they may be studied separately with definite results, and afterward to treat all the remaining variations on the basis of probabilities, using all data from a number of different streams; and to study them in comparison with the normal law of error.

"Among the elements that can be studied separately are the following: Mr. Cory.

"1. The size of the catchment area.

"2. The mean annual run-off per square mile.

"3. The portion of water area and the loss by evaporation from it. This relation is a complex one, and data for determining it are less adequate than could be desired. Nevertheless, some approximations can be reached.

"4. The natural storage in lakes, or in deposits of sand and gravel and other pervious materials. Only approximate results for natural storage can be reached, but as these are found to have a great influence on the required storage, especially at relatively low rates of draft, they must be considered.

"5. Regularity in annual flow. Some streams have comparatively regular flows; in others the variation is much greater. This difference in regularity of flow can be taken into account by finding a coefficient determined from the record of each stream, and bringing this into the statement in such a way that variations from the normal are stated in terms of the 'standard variation'. In this way, records of streams having more regular flows and those having less regular flows may be compared with reference to other matters.

"For the present, all remaining elements of variations of flow of every description will be thrown into one group and studied in connection with the normal law of error."

As a matter of fact, not "all remaining elements of variations of flow of every description" are studied in connection with the normal law of error, but, instead, all the essential elements of flow. As stated under the head, "Data Used," on page 1542:

"The first step is to reduce existing data to the land-area basis, and make a tabular statement showing the average flow per square mile for each catchment area * * *."

That is to say, start with the actually observed run-off reduced to a uniform basis for comparative purposes, which, in itself, is involved and is the result of all five elements specified in the second paragraph, and a great many more. Indeed, with the actually observed run-off once known for any considerable period, the size of the catchment area is ordinarily a matter of indifference; the mean annual run-off per square mile, or otherwise stated, is one summation of many which can be utilized; and the regularity of the flow can at once be observed, or computed in terms of standard variation, or otherwise. The proportion of the drainage area which is water surface, the loss by evaporation from it, and the natural storage in lakes or deposits of sand and gravel, are important, but only primarily as affecting corrections of observed run-off data from several streams, in order to put them on a comparative basis.

The essential thing, then, is that the entire paper (and the method of analysis which it presents) concerns the study of run-off data reduced to a common basis—regardless of how such data may have

Mr. Cory. been collected—with the view of estimating what may be expected from dryer years or years when excessively low flow will occur. Although it is undoubtedly true that the same method could be used for examining precipitation data, and, on page 1572, the author points out that dry years may be classified, with reference to the quantity of rainfall, the quantity of run-off, or the quantities of storage required to maintain certain drafts, nevertheless, as far as the author's presentation as a whole is concerned, actual adjusted run-off is the only thing taken into account and analyzed.

It is thus seen that the third paragraph of the paper as such is misleading, and may give quite a wrong conception of its subject-matter and add considerably to the difficulty of following the line of thought.

As the author states, the first requisite for a successful study of probabilities in run-off and in storage capacities from run-off data, is a sufficient number of facts. To obtain these, the method actually used is to combine the records of several streams reduced to a comparable common basis, thus forming a single series covering a great number of years. The objection to such a method is obvious, but, as a matter of fact, not so great as would seem at first thought. In the light of present knowledge, the combination of records of streams from distant areas having widely different conditions is not justifiable, and a series made up from a number of streams in a relatively small area reflects no changes in climatic conditions which might take place in a long term of years. Mr. Marsh points out that the average daily run-off from the Croton water-shed in the 18 years, 1869-86, was 346 000 000 gal., and in the 18 years, 1887-1904, it was 449 000 000 gal. This difference is 30% of the smaller, and 23% of the larger, figure. As a matter of fact, however, in the treatment proposed in the paper, the safe draft and the storage unit are expressed in terms of the coefficient of variation, which latter does not vary materially for the two periods. If the draft is taken in this way, the variation in the mean annual run-off does not enter into the problem in so marked a manner. The actual draft in any absolute quantity units obviously does, however, and, in applying the results of the analyses explained by Mr. Hazen, one must be careful to realize that if the figure taken for the mean annual flow is not the correct one, the quantity figures will be also in error, the extent of the error when above the critical storage point being, according to the author, about 80% of that in the mean annual run-off (page 1611).

The limitation of the method in this direction, however, does not take away its value for analyzing data of the character under consideration. Obviously, no method of examining data is conceivable which would promise quantity results any more dependable than the figures for mean annual run-off from water-sheds. Serious errors in mean

annual discharge are much less, and can be more easily avoided in a dependable manner, than almost any other kind of errors in hydrographic records. It is undoubtedly true that the relation between the run-off in any one year and the rainfall of that year is impossible to establish within reasonable limits, because of the tremendously complex character of such relationship. The relation between mean annual rainfall over two long periods of time and the mean annual run-off for the same long periods of time, in the case of any given water-shed, however, is a very different matter. The writer does not know just what the facts in this particular are in Eastern, Northeastern, and Southeastern streams, but, in California, where wet and dry cycles run from 8 to 10 years in length, there is no difficulty in judging very clearly from long precipitation records whether or not such low-year run-off and high-year run-off cycles exist. It follows, therefore, that the author's conclusion that his method of studying such an artificial series seems to be the best means now available for finding approximately some of the relations between flows and storage, is absolutely justifiable. Mr. Cory.

On page 1542, it is stated under the heading, "Land Area Only as Basis", that the "published figures are revised by dividing the run-off per square mile from the total area, by one less the proportion of water area." This may be an empirical rule found to be correct for the territory which Mr. Hazen was considering, but otherwise the reason for so doing is not clear. Mr. Marsh's comments on taking equivalent land area rather than gross area seem to be pertinent. On page 1546, it is said that storage is most conveniently stated in days' supply of a maintainable yield. This is evidently true as regards storage for municipal purposes, but it is not true for irrigation and power storage.

It is probable that a great many members began to "read rapidly" —and not understandingly—at the heading, "Probability Paper." The author states that this is:

"* * * paper ruled with lines spaced in accordance with a probability curve, or, as it is otherwise called, the normal law of error. The spacing of the lines for this paper was computed from figures taken from probability curve tables, and arranged so that the line which represents the summation of the probability curve, when plotted on it, is straight."

This is a very short and interesting, but not satisfactory, description. As far as the writer is aware, this is the first time, in engineering literature at least, where probability paper has ever been used, its use suggested, or its preparation even touched on. The short description thus given is, to say the least, not comprehensive, and, indeed, some "arrangement" of the figures taken from probability curve tables must be made in order to obtain the probability paper as given. Just what that arrangement is, the writer has not had time to figure out, and he can well understand that most engineers are

Mr. Cory. not willing to use a new mathematical or graphical tool without at least understanding how it is prepared. The equation of a straight line when plotted on such paper would be interesting. It is obvious that in going into an entirely new field, as the author has in this paper, great conciseness is necessary in order to keep the contribution within reasonable length, but it is hoped that the discussion will bring out a little more information and detail regarding this new probability paper.

The definition of dry years is an excellent idea, and it is well to emphasize the fact that any given percentage dry year must be considered as a type, and not any one actual year.

On page 1574, the author says:

"The importance of making the correction for daily results, when monthly records are used for the basis of calculation, will be realized when it is stated that, for most of the Eastern streams investigated, this correction amounts to more than the allowance for evaporation."

Although this is practically true of the Eastern reservoirs discussed by the author, it is very far from being true in irrigation practice west of the Rocky Mountains. Indeed, in California, the evaporation loss from municipal reservoirs often amounts to more than 25%—being so great sometimes that the hardness of the water at the end of dry cycles is more than doubled. The same applies to the second paragraph on page 1602, in which it is stated that the correction for the quantity of storage in the Croton water-shed, estimated by Mr. Freeman some years ago, deduced by the method of analysis given in the paper, is 30%, "* * * an addition so large that it overshadows all allowances for evaporation, ground-water storage, and other secondary conditions * * *".

For Eastern conditions, where the annual precipitation directly on water areas is approximately equal to the annual evaporation therefrom, the author's methods of handling evaporation, as given on page 1628, is doubtless satisfactory. However, where the evaporation is such a very important factor as it is in the Far West, it is absolutely necessary, in order to avoid very serious mistakes, to eliminate this item on both sides of all equations, and regard it as a part of the total draft, that is, the maintainable draft should be considered as the gross draft, and the net draft should be determined from it and the evaporation, using very short intervals of time and actual water areas for each of the respective time intervals. This is particularly true in the case of irrigation reservoirs, where the draft is not regular throughout the entire year but is concentrated throughout half the year or less, with a maximum daily draft frequently twice that of the average even for the short irrigation season.

As this is the case, in order to make the method general, it seems to the writer that regarding evaporation as one part of the total draft,

just as the net draft is the other part, instead of using the method which the author suggests, is absolutely essential. Mr. Cory.

On page 1596, the paragraph: "It is probable that 2 or more of the 5 years would follow consecutively in one period of several years of low average flow", might be misleading, because the method takes into account this very probability of a succession of dry years.

The paper would be a little clearer and more easily read were parenthetical descriptions of the kind of paper in the various figures indicated, arithmetic-arithmetic, arithmetic-logarithmic, arithmetic-probability, and logarithmic-logarithmic papers being used without stating which is which.

On page 1592, the author states that in the Eastern streams investigated, the annual flow will fall to 55% or less of the normal, once in 100 years. On the other hand, in Western streams it frequently falls to zero for 2 or 3 consecutive years in the case of some important streams. Indeed, the coefficient of variation of the Sweetwater River at the Sweetwater Dam is considerably greater than unity, being 1.38.

On page 1593, the author states:

"It is obvious that the storage required to balance annual fluctuations in flow will increase with the coefficient of variation for the mean annual flows. It is not unlikely that the coefficient of variation may be used as a basis for measuring the storage in such a way that the results will be general, and will apply equally to all the streams within the range covered by this study."

With the view of ascertaining what results would be obtained by applying the method to some of the most variable of the Western streams, the writer examined the run-off data shown in Fig. 38—probably the most variable of any streams in the United States—and that of another series totaling 142 years from another locality (details not given) to see whether the coefficient of variation might be used as a basis for measuring storage. The results were startling, and are given graphically in Fig. 39. Unless it is a case of mere coincidence, which seems improbable, it appears that, not only the method, but this latter curve, can be applied to all streams within the range covered by the writer's study, as well as to all those within that covered by the author.

This is most significant. It indicates that the author has proposed a method by which is obtained a wonderfully effective tool for the use of hydraulic engineers—the figure showing storage units for various values of k for 80, 90, 95, 98, and 99% dry years, and is Fig. 30 extended as low as $k = 0.1$, through the examinations which the writer has just mentioned as having made. In this curve, the storage units equal the standard variation or the coefficient of variation multiplied by the mean flow, and k is the coefficient in the expression:

"Draft = (1 — k × Coef. of variation) = Portion of mean flow used."

Mr. Cory.

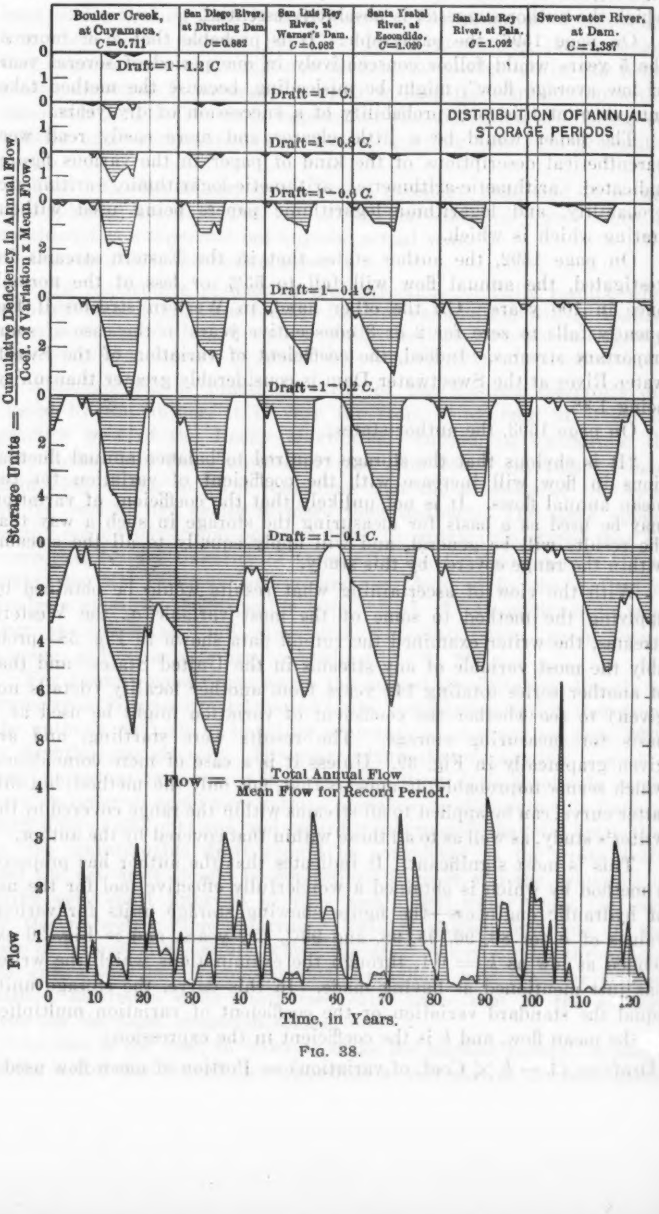


FIG. 38.

It seems premature to suggest that this curve will hold true for streams or sets of streams in general localities, widely separated, and having excessive differences in run-off characteristics. At the same time, the curves do represent very closely the results obtained by the author in the streams which he examined in the New England and neighboring States, and equally well the data from some streams in the very southeastern corner of California and from some in its central portion. In the latter region, the run-off characteristics are quite as different from those in the southern end of California as they are from those selected by the author from New England territory. Mr. Cory.

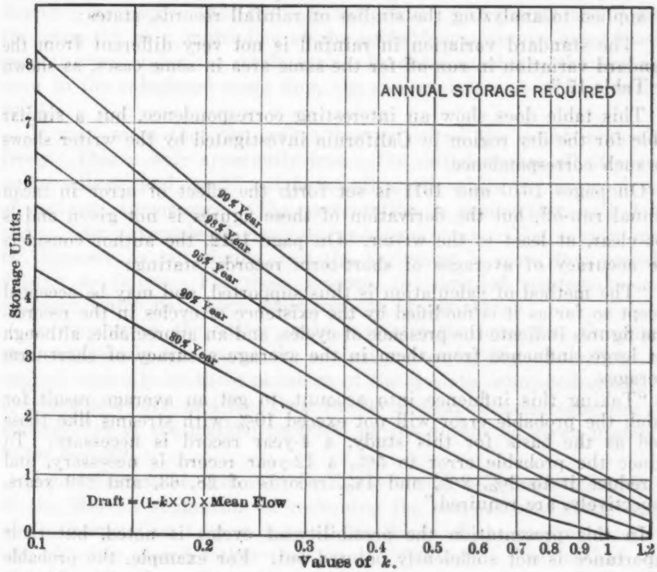


FIG. 39.

Should further investigation show that these curves are really general in their application and that their coincidence in the case of these exceedingly different sets of streams is not a mere matter of chance, the ability to determine very quickly the available draft from any stream with a given quantity of storage, or *vice versa,* by a half day's study of its yearly run-off data, will prove of almost incalculable value to the Profession. It is for this reason and for the desirability of determining quickly the limitations of the method and the propriety of its general application, that the small amount

Mr. Cory. of discussion which the paper has brought out is especially to be regretted.

It is important to remember that to the quantity of storage, as given by the curves in Fig. 39, must be added the actual monthly storage (corrected for daily storage) required to tide over dry seasons in the year, the estimation of which is considered in that part of the paper preceding page 1592. Such monthly storage appears to be about 245 days for the complete development of the Eastern streams investigated by the author, and about 320 days in the California streams investigated by the writer.

On page 1606 the author, having explained that the method can be applied to analyzing the studies of rainfall records, states:

"The standard variation in rainfall is not very different from the standard variation in run-off for the same area in some cases, as shown by Table 17."

This table does show an interesting correspondence, but a similar table for the dry region in California investigated by the writer shows no such correspondence.

On pages 1610 and 1611 is set forth the effect of error in mean annual run-off, but the derivation of these figures is not given and is not clear, at least to the writer. On page 1612, the author considers the accuracy of averages of short-term records, stating:

"The method of calculation is thus supported, and may be accepted except so far as it is modified by the existence of cycles in the records. The figures indicate the presence of cycles, and an appreciable, although not large, influence from them in the average accuracy of short-term averages.

"Taking this influence into account, to get an average result for which the probable error will not exceed 10% with streams like those used as the basis for this study, a 4-year record is necessary. To reduce the probable error to 5%, a 12-year record is necessary, and to reduce it to 3%, 2%, and 1%, records of 28, 63, and 250 years, respectively, are required."

In this presentation the possibility of cycles is noted, but their importance is not sufficiently pointed out. For example, the probable error of the 18-year record, 1887-1904, according to the foregoing, is less than 5%, yet, according to Mr. Marsh's statement in his discussion, this average on the Croton water-shed was 23% greater than the 18-year period, 1867-86. Of course, the variation from the total mean is much smaller, but the safe dependable yield throughout the earlier period of 18 years is obviously the thing, and practically the only thing, of interest to hydraulic engineers in estimating dependable yield, particularly for municipal purposes. With regard to this point, it seems to the writer, as was expressed early in this discussion, that there was no use in hoping to obtain results by analyses of run-off records any more reliable than the figure for the mean annual run-off

Mr. Cory.

itself, and that the possibility of such dry cycles must be examined by a critical analysis of the precipitation records over the same general region, which ordinarily extend back very many times as long as run-off observations.

On page 1616, in the equation for the probable error in the calculated maintainable yield of an Eastern stream above the critical point,

$$\text{Probable error} = \frac{0.55 \times \text{coefficient of variation}}{\sqrt{\text{number of years in record}}},$$

the reason for the use of the figure 0.55 is not clear to the writer. Also, the author's discussion: "Actual Probability of Shortage in Supply", on pages 1634-1636, is not clear, particularly the reason for using 0.8 as a multiplier of the probable percentage error in the general average for the whole series, in order to obtain the probable error in the calculated mean flow, the storage being above the critical point.

It may seem that the writer has criticized the paper somewhat freely. This is only apparently true. The criticisms are all of a very minor nature, and would not be at all worth while if the paper was of ordinary importance. The presentation is remarkably well done, and is a fine example of clearness and conciseness. The subject matter, however, is not simple, and the argument is more than usually difficult to follow. For these reasons, the detailed suggestions and comment which have been made seem to be desirable.

In conclusion, the writer again expresses the hope that the paper, and particularly the use of the revised curves in Fig. 39, will be examined critically by those members of the Society interested chiefly in hydrographic work, and that their results and experiences regarding the applicability of the method proposed by the author be given promptly to the Society.

Mr. Hazen.

Allen Hazen, M. Am. Soc. C. E. (by letter).—There is much force in Mr. Marsh's suggestion of reckoning the evaporation on the water area corresponding to a storage of two-thirds of the maximum quantity of water stored rather than on the full area. The method suggested by Mr. Cory of considering the evaporation as part of the draft is ingenious, and seems to offer substantial advantages for use in dry climates and with long periods of depletion. For Eastern conditions, where, as an annual average, the evaporation is ordinarily less than the rainfall, but considerably exceeds it during a certain period in summer and fall, the method suggested in the paper seems to be better adapted.

There is also the interesting case, occasionally found (not discussed either in the paper or by Mr. Cory), where the water drawn from existing lakes or reservoirs is measured after evaporation has taken place; and, where this is the case, obviously, no further allowance

Mr. Hazen. for evaporation is required in computing the supply that can be utilized from such a source. When, however, it is desired to compare the results with records of flow from streams which have not yet been impounded. allowances for evaporation are necessary in order to estimate the probable natural flow of the stream. This procedure was followed in preparing the Croton, Sudbury, and Wachusett data used in the paper.

After all, the different methods of allowing for evaporation are only different ways of doing the same thing, and, with the same fundamental assumptions, no appreciable difference in results should grow out of any reasonable variation in the method of making the correction.

It is interesting to note that further study of evaporation indicates that, in most cases, early estimates were too high. In other words, the errors in the experimental procedure have usually been in the direction of increasing the apparent quantity of evaporation.

The question of whether the land area or the total area should be used as a basis for stating run-off is one which does not affect in any way the general method proposed by the writer. It makes no difference in the final result which of the two methods is used, so long as that method is kept clearly in mind and is properly applied to the end of the calculation. The land area was selected for use by the writer, for Eastern streams, following Mr. F. P. Stearns' classic discussion of this subject, and he still believes that for these streams this method is, on the whole, rather more convenient. In other cases, it has no advantages.

Mr. Marsh's statement in regard to the mean flow of the Croton for two successive 18-year periods is interesting. Obviously, such periods of low flows must be anticipated from time to time, but how often? The method outlined in the paper gives the basis for a definite answer to this question. The coefficient of variation in annual flows of the Croton River is 0.239. The probable error in any one 18-year period is:

$$\frac{0.239 \times 0.6745}{\sqrt{18}} = 3.8 \text{ per cent.}$$

In the 18-year period from 1869 to 1886, the mean flow was 12.5% less than the record mean for the whole period. Assuming the record mean to be the true mean, the actual error for this 18-year period was 3.3 times as great as the probable error for one 18-year period. From probability tables it appears that 97.4% of all the terms in a series which follows the normal law of error, differ from the mean by less than this amount. The remaining 2.6% of the terms are outside the limits, one-half being above the upper limit and one-half below the lower one. Of all consecutive 18-year series, 1.3%, therefore, would be expected to be less than the true mean by an amount at least as great as that which this Croton series did actually drop

below the record mean of the Croton River. In other words, in any 18-year series, there is 1 chance in 77 that the mean will fall below the true mean by as much or more than the actual reported drop in this case. In an indefinitely long period of years, one such 18-year dry period would be expected once in every 77 years, on an average. The Croton records cover 46 years, to the present time. Within this record are comprised 29 periods, each having 18 consecutive years. Each of these periods had 1 chance in 77 of being below the limit. There were, therefore, 29 chances out of 77 that some one of the 18-year periods would fall short of the record mean by an amount at least equal to the shortage that is reported in this case. The probabilities are almost 2 to 1 that such a shortage will not happen in any given 46-year period. The variation in this record, although it reaches 3.3 times the probable error for any one 18-year series, is one that would have to be expected in one out of every three 46-year periods, and its actual occurrence in this case is an impressive example of the possibility of error in short-term averages. Mr. Hazen.

It is possible, of course, that in this case there may be a long-term cycle variation in rainfall and run-off which may contribute in some measure to the low average, and it may also be mentioned that the greater part of the flow in the first 18-year period was measured as flood discharge over the old Croton Dam, and the greater part of the flow in the second 18-year period was measured in the two Croton Aqueducts on its way to the city; and it may be that, notwithstanding the great care that has been exercised in adjusting the old records, some small residual error may exist and be an element in the reported difference.

Answering Mr. Mills' question, the method proposed is an elastic one and allows the degree of provision against drought to be selected. By working to the 99% year, shortages such as those which Mr. Mills speaks of will be guarded against in 99 years out of 100, though, if the 95% year is selected, a shortage must be anticipated once in 20 years. Therefore, any degree of insurance that is desired can be selected, and the calculation made accordingly.

Mr. Taylor is entirely right in calling attention to the wide difference in run-off and storage conditions between the East and the West, and it is unfortunate that these differences have been so often overlooked, and that such investments as the one which he mentions have been made on inadequate data. On the other hand, as he points out, even in the same case, a properly designed development would have in it great possibilities of good, and would no doubt ultimately prove remunerative.

Mr. Tighe has recorded the origin of the mass-curve, and it may be well to point out that the methods now proposed do not in any way supersede it. On the contrary, the mass-curve method is used as

Mr. Hazen. a basis for computing the storages on which all the tables and diagrams in the paper are based. Practically, however, the mass-curves were not drawn, because it was found that the results might be obtained more easily arithmetically.

Mr. Tighe points out an exception to the rule that, within certain limits, storage on a tributary is substantially as useful as on the main stream. The summer refill water which he mentions, is small in quantity in the dryest years; it is much more abundant in wetter years, when it is not needed. There is no doubt that a small storage on a remote site would have reduced value because of the loss of water in transit in a dry time. With abundant gravel, water lost by percolation may be more important in this case than water lost by evaporation, and, although the former may be expected for the most part to re-appear subsequently on the main stream, it may come at a later time, when the natural flow is otherwise sufficient, and when it is not needed. So far as this is the case, the stored water will be lost and the available supply decreased. The loss of water by percolation extending to and around the intake dam is not taken into account in the method of analysis proposed in the paper. If it were desirable to consider such loss, it would seem best to regard the water lost in this way as a draft, in the same way that Mr. Cory proposes to treat the loss by evaporation.

Mr. Goodrich has suggested a possible explanation of the fact that the curves representing storage and flow data are skew curves, and do not exactly follow the normal law of error. It is his thought that the curve may be composed of several independent curves superimposed on each other. This matter is an extremely important one. It seems to carry great possibilities of improved methods of analysis. The writer earnestly hopes that Mr. Goodrich will find time to develop this method and apply it, and he suggests that this subject would be a most appropriate one for a paper to be presented to the Society.

Mr. Cory's discussion is of fundamental importance. It greatly increases the value of the paper, because it extends the use of the methods to new fields and shows their convenient and useful application. The writer regards the method as more important than the results deduced by it and presented in the paper, and he is particularly interested, therefore, in showing that it is equally applicable in California and in New England, and to problems of municipal water supply, irrigation, and power.

In some cases, at least, the actual numerical figures obtained on a general basis from the records of Eastern streams having the smallest variations in their annual flows are found to be substantially equal to like figures obtained by applying the same methods to Southern California streams having the greatest variations in their annual flows. This coincidence is striking, and it cannot fail to add con-

Mr. Hazen.

fidence to the use of the method for all intermediate conditions, and these include most of the cases that occur. It was the intention to make the methods general in form, so that they would have wide application, but the discovery that both the method and the figures in some of the diagrams do apply to such widely different conditions as those of Southern California is most gratifying.

Fig. 30 is extended far enough to include all the storage ordinarily used, or that will ordinarily be desired on such Eastern streams as serve as the basis of the study; but, in California, where water is relatively more valuable, and where cheap storage sites are found, the economical limit of storage is moved upward many times, as is also shown in Mr. Le Conte's discussion; and many of the larger water projects in successful use in California at the present time will be found to be beyond the limits represented by Fig. 30. The extension of the study to these higher relative storages, therefore, is desirable and necessary. Mr. Cory, using California data, has extended it in this way.

The writer has checked Mr. Cory's results by an independent study of some California data and by extending the Eastern results, and he is satisfied of their substantial accuracy. For the lower relative storages and rates of draft, the Eastern data presented in the paper are adequate to determine the required storage with a fair degree of certainty, and they are better suited to this use than Mr. Cory's California records. On the other hand, the California data for the higher rates of draft are better than the Eastern data, and are to be preferred. The combination diagram (Fig. 39) which Mr. Cory has prepared, therefore, is a safer one to use than could have been deduced from either set of data taken by itself. The fact that the two sets of data produce results which check at the point of junction, both in position and in direction, is most satisfactory.

It is not to be supposed that the storage ratios for the highest rates of draft can be determined with the same degree of accuracy that is reached with the lower ones. With a draft equal to $1 - 0.4C$*, which was the highest limit investigated with the Eastern data, there will be, on an average, a period of about 9 consecutive years in each 20 when there is no overflow. With the records covering 20 or 30 years, and in a few cases somewhat longer periods, this is about as far as the analysis can be carried without sacrifice in accuracy. With the greater draft of $1 - 0.2C$ and $1 - 0.1C$, considered by Mr. Cory, and necessary to include California data and conditions, the normal periods of depletion are greatly extended, until the normal condition of the reservoir becomes one of being only partly full. It is the exception, perhaps the rare exception, when the water reaches the overflow. This condition is actually reached in many California reservoirs

* Mean flow multiplied by 1 less 0.4 coefficient of variation.

Mr. Hazen. at the present time. The greatest normal period of depletion in a 100-year period may rise to 40 or 60 years, or more. It is obvious that records of streams covering only 20 or 30 years are inadequate to establish with certainty the full storage required for these higher rates of draft. The writer, therefore, regards the figures in Mr. Cory's extended diagram (Fig. 39) as approximations, which are the best available at the present time, but are certain to be modified more or less when more extended data become available.

The question may also be considered whether, by mathematical methods and a better analysis of the probabilities of the situation, it may not be possible for some one to reach a better understanding of the matter than the writer has been able to do.

To give Mr. Cory's extended diagram (Fig. 39) full use, and to show its application, not only to California streams, but to many others having higher coefficients of variation than any that were discussed in the paper, the writer presents Figs. 40 and 41, as representing the storage required to maintain the supply in 95% and 98% dry years for California and other similar conditions. These diagrams are based on Mr. Cory's extended diagram (Fig. 39) with a curve for monthly and daily storage based on some studies by the writer of a few California streams. It is the writer's belief that this will be found to fit well enough to be used with advantage in estimating storage in a great variety of climates. It is especially applicable where the variation in annual flows is greater than that for Eastern streams, and where the seasonal fluctuation in flow is also greater. Figs. 40 and 41 also differ from Figs. 33 and 34 in extending much farther upward, to cover streams with greater variations in flow, and with the use of greater relative storages.

The most careful and conservative estimates of public water supplies for Eastern cities in the last decade have been based on assumptions substantially as severe as those corresponding to a 98% dry year. It is the writer's opinion that the 95% year will be more generally applicable, and, on the whole, is a better basis for rating works. If a basis were used corresponding to the 95% year, and it were wrongly assumed that it represented the dryest year to be expected, great damage might follow, but if the limit is recognized and kept always clearly in mind, the likelihood of harm will be largely eliminated. The use of the term, 95% dry year, is in itself a constant reminder that 1 year in 20 will probably be dryer than it, and therefore tends to safety. On the other hand, to limit the rating of works by using the 98% year as a base has the effect of reducing the available supply so much that all of it can be used on an average only once in 50 years, and this will seldom be justified.

The writer regrets Mr. Cory's difficulty in understanding portions of the paper. This may be due to the writer's effort to compress the

Mr. Hazen.

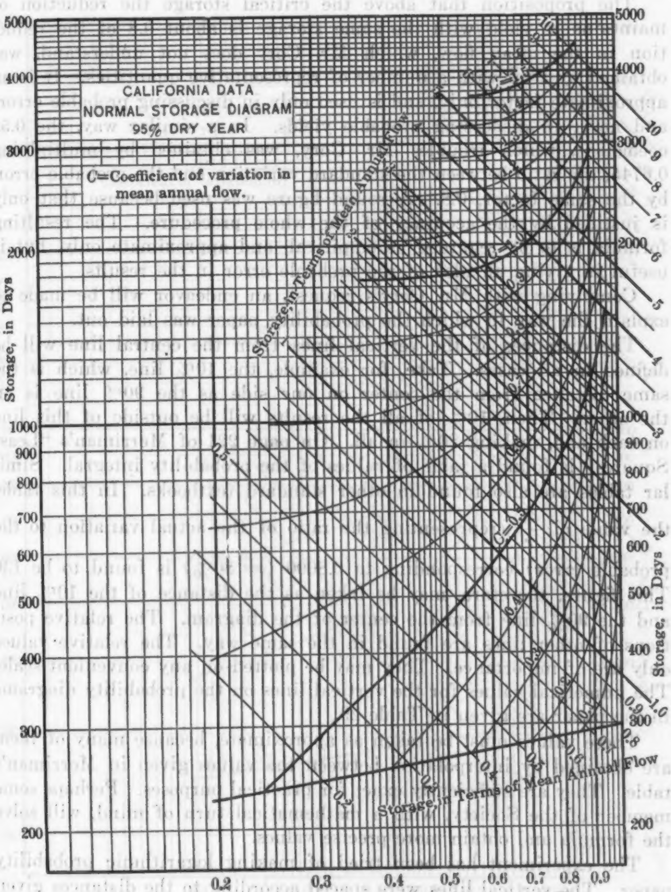


FIG. 40.

Mr. Hazen. material so as to keep the paper within reasonable limits of length, and to avoid making it a treatise on least squares and statistics, and covering ground better covered in textbooks on those subjects.

The proposition that above the critical storage the reduction of maintainable yield with the same storage is about 0.8 of the reduction in the mean flow, which Mr. Cory does not understand, was obtained by trial with a number of representative quantities. It is an approximate factor only, and is used only in discussing probable errors and never in discussing probable yields. In a similar way, the 0.55 coefficient, which troubles Mr. Cory, was obtained by multiplying 0.6745, the ratio between the standard variation and the probable error, by the same factor, 0.8. A round figure was used because that only is justified by the accuracy of the whole procedure. The resulting formula is to be regarded as empirical, and approximate only, but is useful in giving an idea of the probable error in the results.

Complying with Mr. Cory's request, an endeavor will be made to explain the way in which the probability paper was laid out.

The distances of the vertical lines from the central line will be defined in all cases. Take, for example, the 10% line, which is the same distance from the center on one side as the 90% line is on the other. Then, 10% of all the results will be outside of this line on each end, making 20% in all. On page 221 of Merriman's "Least Squares" is found a table of values of the probability integral. Similar tables are also found in other standard textbooks. In this table, the value of $\frac{x}{r}$ (representing the ratio of the actual variation to the probable error) corresponding to 0.8000 (= 80%) is found to be 1.9. This figure, therefore, may be taken as the distance of the 10% line and the 90% line from the center of the diagram. The relative positions of other lines are found in the same way. The relative values only are of importance. They may be plotted on any convenient scale. The numerical values for the vertical lines on the probability diagrams in the paper are given in Table 29.

These values must be taken as approximate, because many of them are obtained by interpolation between the values given in Merriman's table. They are sufficiently exact for practical purposes. Perhaps some member of the Society, with a mathematical turn of mind, will solve the formula and obtain more precise values.

The experiment has been tried of making logarithmic probability paper. The vertical lines were spaced according to the distances given in Table 29, and the horizontal lines were on a logarithmic scale. Numbers plot in a straight line on such paper when their logarithms form a true probability series. Logarithmic probability paper has been found helpful where the degree of skewness in the curve is very great. For the flow and storage data used in the paper, the

Mr. Hazen.

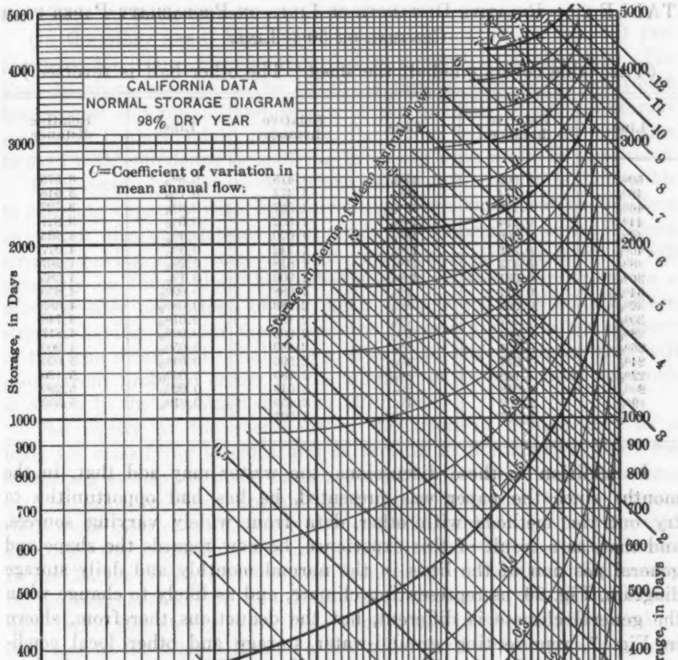


Fig. 41.

Mr. Hazen. arithmetic probability paper has proved more satisfactory. For some other investigations the logarithmic probability paper has seemed to be better.

TABLE 29.—RELATIVE DISTANCES OF LINES ON PROBABILITY PAPER FROM CENTRAL OR 50% LINE.

(This table covers one-half the sheet. The other half is reversed.)

Line.	Relative distance.	Line.	Relative distance.	Line.	Relative distance.
50%	0.000	17%	1.415	0.8%	3.578
48%	0.074	16%	1.474	0.7%	3.646
46%	0.149	15%	1.537	0.6%	3.727
44%	0.224	14%	1.602	0.5%	3.821
42%	0.300	13%	1.670	0.4%	3.933
40%	0.376	12%	1.742	0.3%	4.077
38%	0.453	11%	1.818	0.2%	4.267
36%	0.531	10%	1.900	0.1%	4.585
34%	0.611	9%	1.988	0.09%	4.630
32%	0.693	8%	2.083	0.08%	4.685
30%	0.777	7%	2.188	0.07%	4.748
28%	0.864	6%	2.305	0.06%	4.817
26%	0.954	5%	2.439	0.05%	4.900
24%	1.047	4%	2.596	0.04%	5.000
22%	1.145	3%	2.789	0.03%	5.120
20%	1.248	2%	3.045	0.02%	5.290
19%	1.302	1%	3.450	0.01%	5.550
18%	1.357	0.9%	3.507		

In addition to these discussions, the writer may add that, in the months since the paper was presented, he has had opportunities to try out the methods with other data from widely varying sources, and that, as a result of this experience, he now regards the shape and general position of the lines in the normal monthly and daily storage diagram, Fig. 26, as representing climate, and as likely to change when the general climate is different, but the deductions therefrom, shown in Fig. 7, representing ground-water storage and other local conditions, reflect differences in the several catchment areas, and these must be ascertained for each stream separately. Thus, Fig. 26 is found to apply reasonably well to all the Eastern records from which it is deduced. It probably will be found to apply more or less closely to all streams on the North Atlantic Coast. For English conditions, the required storage would be less than that shown and Fig. 26 would not apply. This is because the English climate has less seasonal variation and the stream flows are steadier. On the other hand, in California, the required monthly storage is greater than that shown by Fig. 26. This figure, therefore, may be accepted only for the general region from which the data were obtained.

For other climatic conditions, the local data could be made to furnish, by the methods described on pages 1546 to 1573, a diagram of normal daily and monthly storage for local use.

Mr. Hazen.

As Fig. 26 serves as the base of Figs. 33 and 34, these diagrams should also be regarded as only strictly applicable to regions where the general climatic conditions do not differ widely from those on the North Atlantic Coast.

On the other hand, Fig. 30, and the extension of it (Fig. 38) prepared by Mr. Cory, may be regarded as general in application. The lines in them are not to be taken as precise, but rather as rough but general and reliable indications of the quantity of annual storage required. Understood in this way, the writer believes that they may be used with confidence as applying to a very wide range in conditions.

It is certainly true, as Mr. Cory points out, that it is impossible to estimate capacity with less percentage error than that in the data on which the calculations rest; and, in connection with Western streams having high coefficients of variation in annual flows, it will generally be found that the probable error of the record mean is a much greater percentage of the mean than is the case with Eastern streams having steadier flows; and the uncertainty as to the amount of the true mean will usually be greater than any difference that will result from probable errors in the number of storage units required, as shown in the combined diagram (Fig. 39).

The utility of the coefficient of variation in annual flow as a basis for classifying streams and as serving as a numerical basis for estimating storage, is made more apparent by further study. Starting with streams in New England and the neighboring States having coefficients most often between 0.20 and 0.25, and always between 0.15 and 0.30, the coefficients are increased to from 0.40 to 0.60 or more in the Middle West, and to more than unity in Southern California. In the neighborhood of San Francisco, streams on the Coast Range with large and steady flows have coefficients in the neighborhood of 0.50, and in the Diablo Range, a few miles farther from the coast, the coefficients are increased to 0.70 or more, and still farther in Southern California to 1.00 and more than 1.00. On the other hand, the Tuolumne River, fed by the melting snows of the Sierras, has a coefficient of only about 0.42. Then, again, with a climate of little variation, as in England, the rather meager data at hand indicate considerably lower coefficients of variation than for any American streams.

When it is realized that the storage required increases in more than a direct ratio with the coefficient of variation, the significance of these figures, as an indication of differences in conditions, and as a numerical basis for estimating the storage required in the different cases, may be realized.

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STATICAL LIMITATIONS UPON THE STEEL REQUIREMENT IN REINFORCED CONCRETE FLAT SLAB FLOORS.*

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Although statics will not suffice to determine the stresses in a flat slab floor of reinforced concrete, it does impose certain lower limits on these stresses. It is the purpose of this paper to inquire into these limiting stresses and to establish their values for comparison with those obtained by current methods of designing floors.

The nature of the limitations imposed by statics is best shown by an illustration. If we are told that three stones weigh 6 lb., this does not establish the weight of any one stone, but it does ensure that the heaviest stone weighs at least 2 lb. Therefore, although we cannot determine the value of the stress at a given section of a flat slab by statics, we can establish a stress intensity for the steel which will certainly be attained and possibly greatly exceeded. Such a quantity would point out the existing danger of overstressing the steel, even though it could not assure us of safety.

Consider the case of an intermediate panel in a floor of the ordinary flat slab type, supported on flare-topped columns, extending indefinitely on all sides, the whole floor subject to full uniform live and dead

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load. This is the case ordinarily presupposed in the design of a floor of this type. For simplicity we will consider all the panels square, and will discuss the equilibrium of that portion of the panel which is heavily outlined in Fig. 1, a quarter-panel, omitting the area over the column top.

The forces acting on this portion of the slab are its own weight and the live load, both uniformly distributed over its surface; the vertical shear on the curved section, A; and the couples (in vertical planes) on the sections, A, B, C, D, and E.

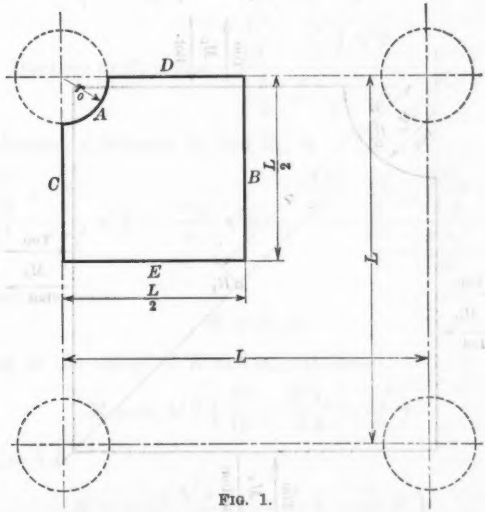

FIG. 1.

The assumption that there is no shear on the sections, B, C, D, and E, is not made arbitrarily, however reasonable it may be. Not only is it a reasonable assumption, but it is the most favorable that can be made with reference to low stresses. For, if any shear existed on one of these sections, in such a way as to lessen the other stresses, this same shear acting in a reverse direction on the adjacent quarter-panel would increase correspondingly the stresses there, and we would have to give our attention to that quarter-panel which involves the greatest stresses. Obviously, the most favorable supposition is that all quarter-panels are equally effective load carriers, and this means, as just stated, absence of shear from the four straight sides.

The only outright assumption to be used as a basis for the inquiry is that the shear on the curved section, A, is uniformly distributed. The reasonableness of this assumption is obvious, and besides, a considerable variation in the shear on A involves only a slight error in the results we shall obtain, for the only use made of this assumption is in locating the resultant of the vertical shearing forces.

If the unit live and dead load is w pounds per square foot, the total load on the quarter-panel is

$$R_1 = w\left(\frac{L^2}{4} - \frac{\pi r_0^2}{4}\right).$$

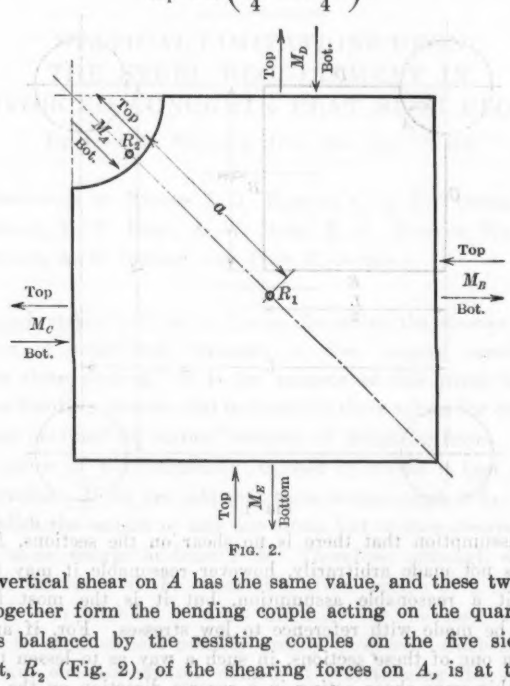

FIG. 2.

The vertical shear on A has the same value, and these two vertical forces together form the bending couple acting on the quarter-panel, which is balanced by the resisting couples on the five sides. The resultant, R_2 (Fig. 2), of the shearing forces on A, is at the center of gravity of the 90° arc at A, which is on the diagonal line from the column to the opposite corner of the quarter-panel, and is distant from the center of the column

$$\frac{2\sqrt{2}}{\pi} r_0.*$$

* See any Engineers' Handbook.

The resultant of the loads passes through this same diagonal line. Its distance, z, from the center of the column is obtained as follows, taking moments about the column center:

	Area.	Distance.	Moment.
Square.....	$\frac{L^2}{4}$	$\times \frac{L}{4}\sqrt{2}$	$= \frac{L^3}{16}\sqrt{2}$
Quadrant...	$\frac{\pi r_0^2}{4}$	$\times \frac{4\sqrt{2}}{3\pi} r_0$	$= \frac{r_0^3}{3}\sqrt{2}$
Difference..	$\frac{L^2}{4} - \frac{\pi r_0^2}{4}$	$\times z$	$= \left(\frac{L^3}{16} - \frac{r_0^3}{3}\right)\sqrt{2}$

$$\text{Distance to } R_1 \text{ is } z = \frac{\left(\frac{L^3}{16} - \frac{r_0^3}{3}\right)\sqrt{2}}{\frac{L^2}{4} - \frac{\pi r_0^2}{4}}.$$

The distance, a, between R_1 and R_2, is

$$a = \frac{\frac{L^3}{16} - \frac{r_0^3}{3}}{\frac{L^2}{4} - \frac{\pi r_0^2}{4}}\sqrt{2} - \frac{2 r_0}{\pi}\sqrt{2} = \frac{\frac{L^2}{16} - \frac{r_0^3}{3} - \frac{L^2 r_0}{2\pi} + \frac{r_0^3}{2}}{\frac{L^2}{4} - \frac{\pi r_0^2}{4}}\sqrt{2}.$$

We may now write

$$M = R_1 a,$$

or, putting in the values of R and a, just found,

$$M = w\sqrt{2}\left(\frac{L^3}{16} - \frac{L^2 r_0}{2\pi} + \frac{r_0^3}{6}\right).$$

Let $r_0 = k L$

$$M = w L^3 \frac{\sqrt{2}}{16}\left(1 - \frac{8}{\pi}k + \frac{8}{3}k^3\right)$$

where M is the couple made up of the load and the supporting shear in the vertical diagonal plane through R_1 and R_2.

The component of this couple about a horizontal axis parallel to the side, B, is

$$M_x = \frac{w L^3}{16}(1 - 2.55\,k + 2.67\,k^3)$$

or

$$M_x = K w L^3$$

where

$$K = \frac{1 - 2.55\,k + 2.67\,k^3}{16}.$$

M_y, the component of M about a horizontal axis parallel to D, evidently has the same value as M_x.

Values of K for ordinary values of k are given in the diagram, Fig. 3.

Having found the attacking couple, M (or its two components M_x and M_y), there remains but to point out that the resisting moments

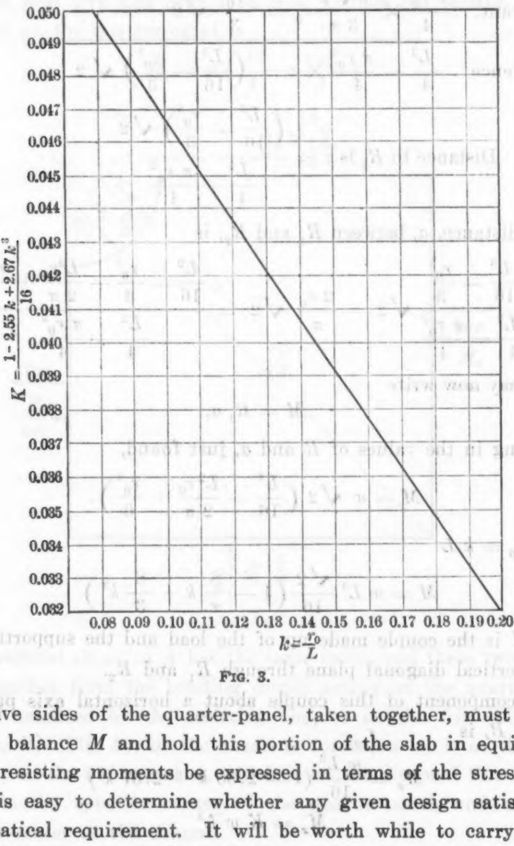

Fig. 3.

on the five sides of the quarter-panel, taken together, must be sufficient to balance M and hold this portion of the slab in equilibrium. If these resisting moments be expressed in terms of the stress in the steel, it is easy to determine whether any given design satisfies this purely statical requirement. It will be worth while to carry the inquiry a little farther and apply the results already obtained to a few simple types of reinforcement.

I.—Perhaps the simplest type consists of rods parallel to the sides of the square panels, at the top of the slab where they cross A, C, and D (Fig. 4), and at the bottom where they cross B and E. Let A_s be the total area of all the rods in both directions in a panel, d' the vertical distance from the center of the steel to the center of compression of the concrete, and f_s the unit stress in the steel. The area of cross-section of the steel in Section B is $\frac{A_s}{4}$, the total tension is $\frac{A_s f_s}{4}$, and the resisting moment on B is

$$M_B = \frac{A_s}{4} d' f_s.$$

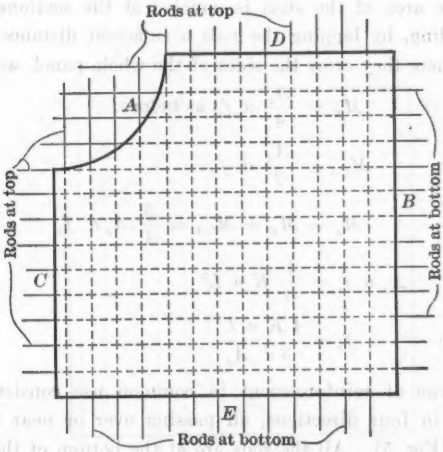

FIG. 4.

Similarly, the resisting moment on C, including the component about an axis parallel to C, of the moment on A, is

$$M_{CA} = \frac{A_s}{4} d' f_s.$$

We have, from statics, the condition of equilibrium that the sum of the moments about any axis must be zero. Applying this principle to the present case, and taking a horizontal axis parallel to C, we get

$$M_x = M_B + M_{CA}$$

$$= \frac{A_s}{2} d' f_s = K w L^3$$

or,

$$A_s d' f_s = 2 K w L^3$$

$$f_s = \frac{2 K w L^3}{d' A_s}.$$

It should be borne in mind that this is the lowest limit of the value of A_s for a given maximum f_s consistent with the fundamental principles of statics; and the maximum stress in the steel for a given A_s will not be as low as f_s, except under the conditions already assumed (and they are the most favorable possible), and under the further assumption that all the steel at all the five sections, which constitute the periphery of the quarter-panel, is uniformly stressed. If the stress intensity is less than f_s at any point, it must necessarily be greater than f_s at some other point. The stated value of f_s is certainly reached and may be greatly exceeded.

II.—If the area of the steel is doubled at the sections subject to negative bending, by lapping the rods a sufficient distance at the top of the slab where they cross the sides of the whole panel, we may write

$$M_B = \frac{A_s}{4} d' f_s, \text{ as before,}$$

but now,

$$M_{CA} = \frac{A_s}{2} d' f_s$$

$$M_x = M_B + M_{CA} = \frac{3}{4} A_s d' f_s$$

$$A_s d' f_s = \frac{4}{3} K w L^3$$

$$f_s = \frac{4 K w L^3}{3 d' A_s}.$$

III.—A type of reinforcement in common use consists of equal belts of rods in four directions, all passing over or near the tops of the columns (Fig. 5). All the rods are at the bottom of the slab midway between the columns and at the top over the columns. The pull in the rods piercing the side, B, of the quarter-panel, amounts to $\frac{f_s A_b}{2}$ from the half of the Belt (1) and $\frac{f_s A_b}{\sqrt{2}}$ from the halves of the Belts (2) and (3), where A_b is the area of cross-section of one belt. Both of these are normal to the side, B. The resisting moment on B is then

$$M_B = \left(\frac{A_b}{2} + \frac{A_b}{\sqrt{2}}\right) d' f_s = 1.207 A_b d' f_s.$$

Similarly, for the steel over the column

$$M_{CA} = \left(\frac{A_b}{2} + \frac{A_b}{\sqrt{2}}\right) d' f_s = 1.207 A_b d' f_s.$$

Then $$M_x = M_B + M_{CA} = 2.41\ A_b\ d'\ f_s$$

$$A_b = \frac{K\ w\ L^3}{2.41\ d'\ f_s}.$$

The total cross-sectional area of the four belts is

$$A_s = 4\ A_b = \frac{1.66\ K\ w\ L^3}{d'\ f_s}$$

$$A_s\ d'\ f_s = 1.66\ K\ w\ L^3$$

$$f_s = \frac{1.66\ K\ w\ L^3}{d'\ A_s}.$$

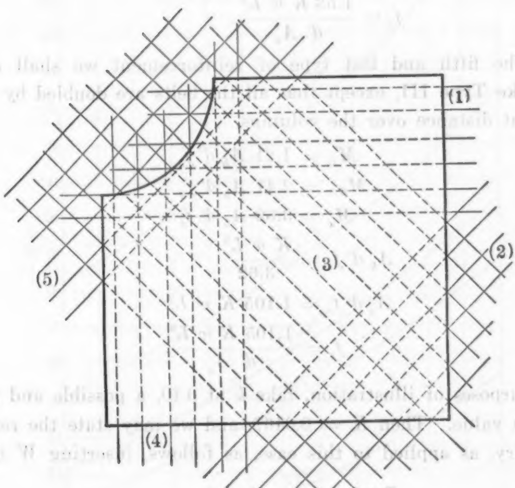

FIG. 5.

The value of A_s for this type appears to be less than that for Type I, other things being equal, but as half the rods here are 1.41 times as long, neither type shows economy over the other in weight of steel.

IV.—A fourth type is like Type III except that the diagonal rods are doubled over the column by lapping, and the rectangular or short-way belts remain in the bottom of the slab even where they pass over the columns.

Using the same notation as before, we have

$$M_B = 1.21\, A_b\, d'\, f_s \text{ as in Type III,}$$

$$M_{CA} = \frac{2\, A_b}{\sqrt{2}}\, d'\, f_s = 1.41\, A_b\, d'\, f_s$$

$$M_x = M_B + M_{CA} = 2.62\, A_b\, d'\, f_s$$

$$A_b = \frac{K\, w\, L^3}{2.62\, d'\, f_s}$$

$$A_s = \frac{1.53\, K\, w\, L^3}{d'\, f_s}$$

$$A_s\, d'\, f_s = 1.53\, K\, w\, L^3$$

$$f_s = \frac{1.53\, K\, w\, L^3}{d'\, A_s}.$$

V.—The fifth and last type of reinforcement we shall examine here is like Type III, except that all the belts are doubled by lapping a sufficient distance over the columns.

$$M_B = 1.21\, A_b\, d'\, f_s$$

$$M_{CA} = 2.41\, A_b\, d'\, f_s$$

$$M_x = 3.62\, A_b\, d'\, f_s$$

$$A_b\, d'\, f_s = \frac{K\, w\, L^3}{3.62}$$

$$A_s\, d'\, f_s = 1.105\, K\, w\, L^3$$

$$f_s = \frac{1.105\, K\, w\, L^3}{d'\, A_s}.$$

For purposes of illustration, take k at 0.10, a possible and perhaps a common value. Then $K = 0.0467$, and we may state the results of this inquiry, as applied to this case, as follows, inserting W in place of $w\, L^2$.

Type.	$A_s\, d'\, f_s$.
I.	$0.0935\, W\, L$ or $\dfrac{W\, L}{10.7}$
II.	$0.0623\, W\, L$ or $\dfrac{W\, L}{16}$
III.	$0.0775\, W\, L$ or $\dfrac{W\, L}{12.9}$
IV.	$0.0713\, W\, L$ or $\dfrac{W\, L}{14}$
V.	$0.0517\, W\, L$ or $\dfrac{W\, L}{19.3}$.

If $A_s\, d'\, f_s = \frac{W\, L}{X}$, the values of X are given for the five types in the diagram, Fig. 6.

There remains but to point out the bearing of all this on current practice in the design of flat slabs. It cannot be recalled too often that the results obtained in this paper are based solely on statics and

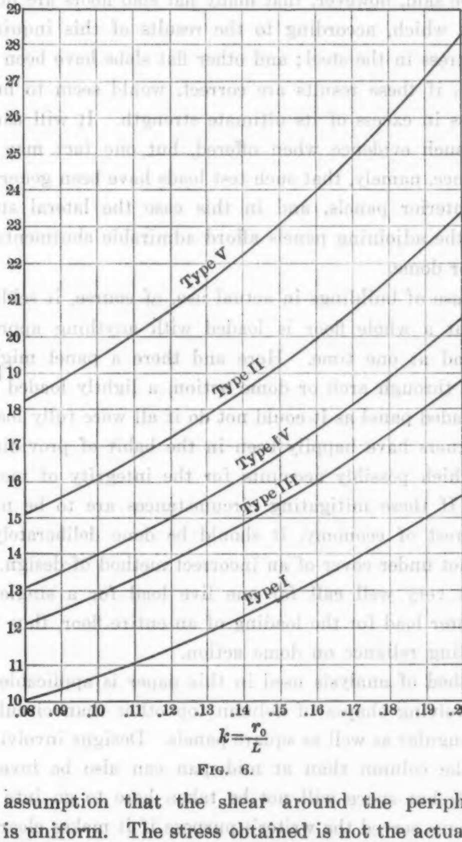

FIG. 6.

the single assumption that the shear around the periphery of the column-top is uniform. The stress obtained is not the actual maximum stress in the steel, but is the lowest value which this stress can possibly have under the loads and conditions assumed. The actual stresses

are determined by the elastic properties of the materials, and their computation, to say the least, is intricate. With whatever industry the powers of higher mathematics and the mysteries of Grashof's formulas and Poisson's ratio may be invoked, they cannot justify a result for the maximum stress in the steel smaller than the limiting value determined in this paper.

It will be said, however, that many flat slab floors are now standing under loads which, according to the results of this inquiry, imply a very high stress in the steel; and other flat slabs have been tested with loads which, if these results are correct, would seem to bring on the steel a stress in excess of its ultimate strength. It will usually suffice to discuss such evidence when offered, but one fact may be pointed out in advance, namely, that such test loads have been generally applied to single interior panels, and in this case the lateral strength and rigidity of the adjoining panels afford admirable abutments for a concrete arch or dome.

In the case of buildings in actual use, of course, it seldom, if ever, happens that a whole floor is loaded with anything approaching its full live load at one time. Here and there a panel might be fully loaded, but, through arch or dome action, a lightly loaded panel helps a heavily loaded panel as it could not do if all were fully loaded. Then again, designers have happily been in the habit of providing a factor of safety which possibly accounts for the integrity of many existing buildings. If these mitigating circumstances are to be made use of in the interest of economy, it should be done deliberately and with eyes open, not under cover of an incorrect method of design. Specifications might very well call for one live load for a single panel and another lighter load for the loading of an entire floor, thus recognizing and permitting reliance on dome action.

The method of analysis used in this paper is applicable, of course, to cases involving shapes of column top other than circular in plan, and to rectangular as well as square panels. Designs involving a deeper slab near the column than at mid-span can also be investigated by this method; but space will not be taken here to go into that. The paper will have served the writer's purpose if it makes clear the nature of the limitations set by statics upon the load-carrying capacity of flat slab floors with reference to the stress in the steel, and sufficiently

illustrates the application of the principles involved to facilitate their extension to other cases that may arise.

In closing, the writer asks those who criticize this paper not to confine themselves to citing evidence appearing to conflict with his findings, but also to point out any errors they may find in his premises or reasoning.

DISCUSSION

Mr. Mensch. L. J. MENSCH, M. AM. SOC. C. E. (by letter).—This paper invites a timely discussion of the design of the so-called flat slab floors. It seems that the best firms in America have a fairly good knowledge, based on expensive tests, of the maximum stresses which enter into the design of such plates, yet they cannot convince the conservative engineer or architect that their designs are not a gamble, and that many of their structures will not show serious defects sooner or later. To the writer's knowledge, the responsible firms which make a specialty of designing and building such floors do not use higher stresses than the author claims to find by static methods. His paper, however, will not advance our knowledge of the design of such floors, as he assumes a certain relation of positive and negative moments, and fails to prove that they may exist.

The writer will endeavor to develop a rational method of designing flat floors, practically without the help of the higher mathematics. For this purpose he will consider the floor to consist of two systems of slabs, at right angles to each other and continuous over the supports. As shown in Fig. 7, each slab carries one-half of the panel load of each panel, and is of a width equal to the distance from center to center of columns. The theory of continuous beams will be applied to a few of the most important cases.

Case I.—Fig. 8.—Continuous Slab on Four Supports; Only the Center Panel Loaded, by the Triangular Load, $\frac{W}{2} = \frac{wl^2}{2}$.—

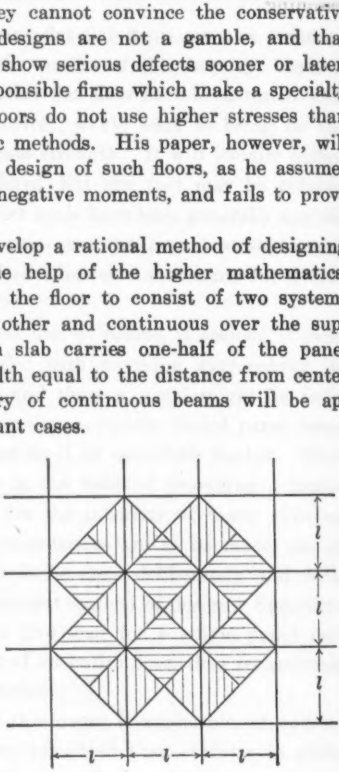

FIG. 7.

One easily finds the reaction at $A = -\frac{1}{32}\,W$, and the reaction at $B = \frac{9}{32}\,W$.

The moment over the support, $B = Al = -\frac{1}{32}\,Wl$.

The moment in the center of the loaded panel $= A\left(l + \frac{l}{2}\right) + B\,\frac{l}{2} - \frac{W}{4}\,\frac{l}{6} = \frac{5}{96}\,Wl = \frac{1}{19.2}\,Wl.$

Case II.—Fig. 9.—Continuous Slab on Four Supports; Only the Outside Panels Loaded. Load on Each Panel, $\frac{W}{2}$.—

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The reaction at $A = \frac{7}{32} W$.

The reaction at $B = \frac{9}{32} W$.

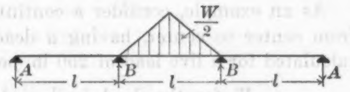

FIG. 8.

The moment in the center of the loaded panels $= \frac{13}{192} Wl = \frac{1}{14.8} Wl$.

The moment at B and for the entire center panel $= Al - \frac{W}{2} \frac{l}{2} = -\frac{Wl}{32}$.

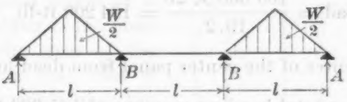

FIG. 9.

Case III.—Fig. 10.—Continuous Slab on Four Supports; Each Panel Loaded by a Triangular Load, $\frac{W}{2}$.—

The reaction at $A = \frac{3}{16} W$.

The reaction at $B = \frac{9}{16} W$.

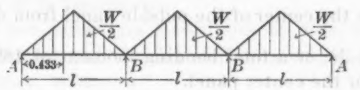

FIG. 10.

The maximum moment at 0.433 l from $A = \frac{Wl}{18.5}$.

The moment over $B = Al - \frac{W}{2} \frac{l}{2} = -\frac{Wl}{16}$.

The moment in the center of the center panel

$$= B \frac{l}{2} - \frac{W}{4} \frac{l}{6} = -\frac{Wl}{16} = \frac{Wl}{48}.$$

Case IV.—As in Case I, but With an Infinite Number of Panels; and the Triangular Load, $\frac{W}{2}$, *Applied to the Center Panel.*—The bending moments, over the support, as well as in the center of the loaded panel, are the same as given under Case I.

Case V.—As in Case III, but With an Infinite Number of Panels.—The bending moments over the central supports $= \frac{W\,l}{19.2}$. The bending moments in the center of the center panel $= \frac{W\,l}{32}$. In this case the outside panels have about the same maximum moments in the center and over the second support as the outside panels in Case III. The maximum moments in the second panels from the outside $= \frac{W\,l}{35.5}$, and those over the third supports $= \frac{W\,l}{22.4}$.

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As an example, consider a continuous slab on four supports, 20 ft. from center to center, having a dead load of 125 lb. per sq. ft., and calculated for a live load of 250 lb. per sq. ft.

W for the dead load $= 125 \times 400 = 50\,000$ lb.
W for the live load $= 250 \times 400 = 100\,000$ "

For Case I, the moment in the center of the center panel from live load $= \frac{100\,000 \times 20}{19.2} = 104\,200$ ft-lb. For Case III, the moment in the center of the center panel from dead load $= \frac{50\,000}{48} \times 20 = 20\,800$ ft-lb., or a total bending moment of 125 000 ft-lb.

For Case II, the moment in the center of the outside panel from live load $= \frac{100\,000 \times 20}{14.8} = 135\,000$ ft-lb. For Case III, the moment in the center of the outside panel from dead load $= \frac{50\,000}{18.5} \times 20 = 54\,000$ ft-lb., or a total bending moment of 189 000 ft-lb., or 50% greater than for the center panel.

The moment over the support, B, is greatest for Case III, and equals $\frac{150\,000 \times 20}{16} = 188\,000$ ft-lb., or the slab must be just as strong over the supports as in the center.

The entire inside panel is subject to a negative moment from the live load for Case II, which moment $= \frac{100\,000 \times 20}{32} = 62\,500$ ft-lb., which in the center of the panel is reduced by the dead load moment of $\frac{50\,000 \times 20}{48} = 20\,800$ ft-lb. These high negative moments cannot be neglected by a responsible designer. The author and most of the designers seem to consider only Case V, for which the moment in the center of the center panel $= \frac{150\,000 \times 20}{32} = 94\,000$ ft-lb., and the moment over the supports of the center panel $= \frac{150\,000 \times 20}{19.2} = 156\,000$ ft-lb., or considerably less than in the foregoing cases, or in the outside panels of Case V.

The moments heretofore given are for the entire slab, having a width $= l$. The assumption made by the author is that the stresses are distributed uniformly. By dividing the width of the panel into, say, ten units, it can be shown that a parabolic distribution of the entire moment, for the sizes of caps ordinarily used, is as near an assumption as can be made. See Fig. 11. Then the moments in the middle portion of the slab are 25% greater than the average moments and 2.5 times

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as great as at the edge of the slab. The foregoing computations are only adaptable to flat slabs with caps which have universal joints over the centers of the columns. This is not the case in practice; on the contrary, the columns are well connected with the floors, and nearly always have a larger moment of inertia than the floor slab of a total width $= l$. Besides, the height of the columns in most cases is considerably less than the span of the floor, and, therefore, the columns will offer a greater resistance to a change of angle over the supports than can be expected from the continuous action of the floor when only one panel is loaded. As a rule, there are columns on top as well as below the floor, and these will increase further the arch action which is to be considered.

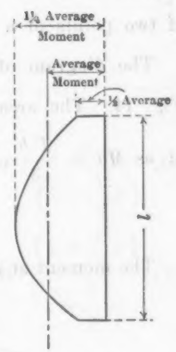

FIG. 11.

It will now be assumed that two panels are loaded side by side, as in Fig. 12, and that the two columns and the flat slab of a width, l, form a hingeless arch construction, as shown in Fig. 13.

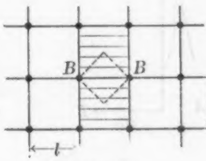

FIG. 12.

To facilitate the calculation, it will be assumed that the columns cannot change their inclination at A. Two unknown values have to be found, the thrust, T, and the moment, Ma.

Let I = the moment of inertia of the column;

I_1 = the moment of inertia of the floor slab of the width, l;

h = the story height;

l = the distance from center to center of columns;

$$n = \frac{h\, I_1}{l\, I}.$$

It is known, from the theory of beams, that the column, AB, which is subject only to a force, T, and a moment, Ma, must have its point of inflection at $\frac{1}{3}\, h$ from A, because the column cannot change its angle at A, as has been assumed. It immediately follows that $Ma = \frac{T\, h}{3}$.

To find the unknown value, T, we have to consider that the inclination of the column at B with the vertical must be equal to the inclination of the slab with the horizontal line at B, and that, by a

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well-known rule of mechanics, the change of angle between tangents of two points of a beam equals $\frac{1}{E\,I}$ (area of diagram of moments).

The diagram of moments for the columns and slab is shown in Fig. 14. The area of the diagram for a column $= T\,h\,\frac{h}{2} - Ma\,h$, or, as $Ma = \frac{T\,h}{3}$, the area $= \frac{T\,h^2}{6}$

$$\text{and } a = \frac{1}{E\,I} \times \frac{T\,h^2}{6} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The moment at $B = T\,h - Ma = \frac{2}{3}\,T\,h.$

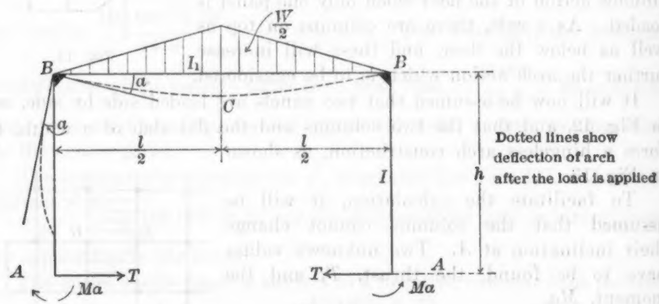


FIG. 13.

It is easily found that the moment at any point between B and C

$$= \frac{Wx}{4} - \frac{W}{3}\,\frac{x^3}{l^2} - \frac{2}{3}\,T\,h \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The change of angle between the points, B and C, is again a, and equals $\frac{1}{E\,I_1}$ (area of diagram of moments between BC),

$$= \frac{1}{E\,I_1}\left(\frac{W}{4} \times \frac{1}{2} \times \frac{l^2}{4} - \frac{W}{3\,l^2} \times \frac{1}{4} \times \frac{l^4}{16} - \frac{2}{3}\;T\,h \times \frac{l}{2}\right)$$

$$a = \frac{1}{E\,I_1}\left(\frac{5}{192}\;W\,l^2 - \frac{T\,h\,l}{3}\right) \ldots\ldots (3)$$

Combining Equations 1 and 3, $\frac{1}{E\,I} \times \frac{T\,h^2}{6} = \frac{1}{E\,I_1}\left(\frac{5}{192}\;W\,l^2 - \frac{Th}{3}\,l\right)$

$$\frac{1}{I}\;\frac{T\,h^2}{6} + \frac{1}{I_1}\;\frac{T\,h\,l}{3} = \frac{1}{I_1}\;\frac{5}{192}\,W\,l^2$$

$$\frac{T\,h\,l}{3\,I_1}\left(1 + \frac{h}{l}\;\frac{I_1}{I} \times \frac{1}{2}\right) = \frac{1}{I_1}\;\frac{5}{192}\;W\,l^2$$

Mr. Mensch.

or $$Th = \frac{5}{64} W l \frac{1}{1 + \frac{1}{2} n} \quad \text{.......................(4)}$$

or $$Ma = \frac{5}{192} W l \frac{1}{\left(1 + \frac{1}{2} n\right)} \quad \text{.......................(5)}$$

or $$M_B = \frac{5}{96} W l \frac{1}{\left(1 + \frac{1}{2} n\right)} = \frac{1}{19.2} W l \frac{1}{\left(1 + \frac{1}{2} n\right)} \quad .(6)$$

From Equation 2, $$M_C = \frac{W}{4} \frac{l}{2} - \frac{W}{3 l^2} \times \frac{l^3}{8} - M_B = \frac{1}{32} W l \frac{\left(1 + \frac{4}{3} n\right)}{1 + \frac{1}{2} n} \quad .(7)$$

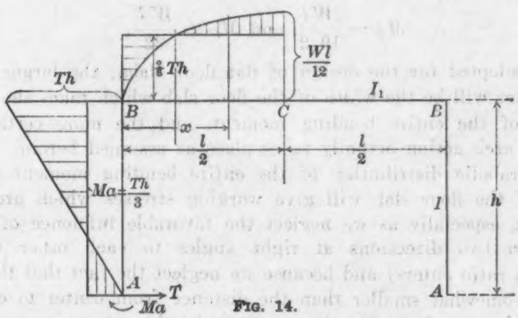


FIG. 14.

These are practically the same equations as found for Case V, modified only by the value of n. It will be assumed that the floor in the former example is 9 in. thick, and that the columns are 24 in. in diameter.

$l = 20$ ft.

$h = 12$ ft.

$t = 9$ in.

$$I_1 = 240 \times \frac{9^3}{12} = 14\,580.$$

$$I = 24^4 \frac{\pi}{64} = 162\,000,$$

$$n = \frac{h}{l} \frac{I_1}{I} = \frac{12}{20} \times \frac{14\,580}{162\,000} = \frac{1}{18.5}.$$

Where columns are also on the top of the floors, n must be taken as one-half of the value thus found, or, in this case, $n = \frac{1}{37}$. n must

Mr. Mensch. be further reduced because the adjacent floor slab increases the restraining influence, and a value for n of about $\frac{1}{40}$ is probably correct for this case. The smallest column which could be used in this case to carry the load of 150 000 lb. would be 15 in. in diameter, or $I = 25\,000$, and $n = \frac{1}{2.86}$. From Equations 6 and 7 we obtain

$$\begin{array}{llcccccc}
\text{for } n = & & 1 & \frac{1}{2} & \frac{1}{4} & \frac{1}{10} & \frac{1}{40} & \frac{1}{\infty} \\[6pt]
M_B = & W\,l \times & \frac{1}{28.6} & \frac{1}{24} & \frac{1}{21.6} & \frac{1}{20.1} & \frac{1}{19.7} & \frac{1}{19.2} \\[6pt]
M_C = & W\,l \times & \frac{1}{20.6} & \frac{1}{24.1} & \frac{1}{27.1} & \frac{1}{29.7} & \frac{1}{31.4} & \frac{1}{32}
\end{array}$$

In most cases n is a very small value, and the moments of

$$M_B = \frac{W\,l}{19.2}, \text{ and } M_C = \frac{W\,l}{32},$$

may be adopted for the design of flat floor slabs; the larger the cap the greater will be the width of the floor slab which takes the greatest portion of the entire bending moment, and the more certain is it that the arch action actually takes place as assumed herein.

A parabolic distribution of the entire bending moment over the width of the floor slab will give working stresses which are on the safe side, especially as we neglect the favorable influence of stresses acting in two directions at right angles to each other (wherein Poisson's ratio enters) and because we neglect the fact that the actual span is somewhat smaller than the distance from center to center of columns, but much greater than assumed by the author.

The bending moment of $\frac{W\,l}{19.2}$ (W being the total live load for inside panels and the total live and dead load for outside panels) is not taken up by the columns alone, but is divided between the columns and the floor slabs of the adjacent unloaded panels, and may be found as follows:

$$M_B \text{ for column} = \frac{M_B}{n+1}$$

$$M_B \text{ for adjacent slab} = M_B\,\frac{n}{n+1},$$

where $n = \frac{h}{l}\,\frac{I_1}{I}$, without any reduction. The portion of the moment taken by the columns must again be divided in proportion to their respective moments of inertia between the lower and upper columns. In this example, $\frac{W\,l}{19.2} = 104\,000$ ft-lb.

Mr. Mensch.

Assuming that one-half is taken by the lower columns and one-half by the upper columns, each column must be calculated for a bending moment of 50 200 ft-lb., which will cause a maximum compression in the 24-in. columns of at least 400 lb. per sq. in., which stresses, the writer is sorry to state, are generally neglected, in building flat floor constructions, even by the most successful firms. The fact is that, although the columns are under high compressive stresses in the lower stories of buildings, under test loads, cracks in columns were noticed before they were detected in the slabs, proving conclusively that this arch action exists, and that the columns must be calculated for bending.

There is still another proof that this arch action exists. The deflection of a slab floor can be calculated by introducing Equation 4 into Equation 2 and integrating twice; then, for $x = \frac{l}{2}$ we obtain

$$\text{the deflection in the center of a side} = \frac{7}{3\,840} \times \frac{W l^3}{E I_1} \times \frac{1 + n}{1 + \frac{1}{2} n}.$$

Substituting for $I = l \frac{t^3}{12}$ and neglecting n, the deflection

$$= \frac{1}{45.7} \times \frac{W l^2}{E t^3} \dots\dots\dots\dots\dots\dots\dots (8)$$

This result is in inches when l and t are in inches.

It can also be shown that a strip forming the center of a square which is subject to a moment of $\frac{W\,l}{36.4}$ in the center and $\frac{W\,l}{64}$ at its ends must deflect one-half the amount given in Equation 8, and we obtain

$$\text{the maximum deflection in the center of the square as} = \frac{1}{30.5} \frac{W\,l^2}{E\,t^3} \,.\,(9)$$

Introducing for E the value, 1 500 000, the deflections thus calculated are borne out by tests more closely than for any T-beam test which has come to the writer's notice.

To sum up: A flat floor slab should be calculated for a negative

$$\text{bending moment over the support of } \frac{w\,l^2}{15.3} \dots\dots\dots\dots\dots\dots (10)$$

per linear foot; and for a negative bending moment in the center of

$$\text{the sides of } \frac{w\,l^2}{38.4} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (11)$$

$$\text{It should be calculated for a positive moment of } \frac{w\,l^2}{25.6} \dots\dots\dots (12)$$

Mr. Mensch. per linear foot near the side of the square; and for a moment of $\frac{w\,l^2}{64}$.(13) per linear foot near the center of the square, provided the columns are calculated for a moment of $\frac{W\,l}{19.2}$.

It remains yet to be shown how flat floor slabs are to be calculated when the columns are arranged in the corners of rectangles instead of squares, as shown in Fig. 15.

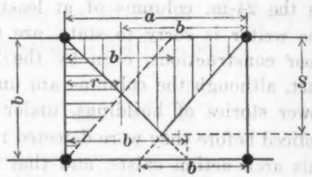


FIG. 15.

Where $\frac{a}{b}$ is less than 1.5, it is the writer's practice to calculate the long span for a width of $2s$, and to assume $W = w \times a \times 2s$, and to calculate the short span for the width, $2r$, and the corresponding $W = w \times b \times 2r$.

For a greater ratio of a to b, it is best to calculate the long span for the total moments of $\frac{Wl^2}{12}$ and $\frac{Wl^2}{24}$, over the supports and in the center of the span, when $W = w \times a \times b$, and to calculate the short span as before for $W = w \times b \times 2r$.

As in all other continuous girder constructions, unequal settlements of the supports will produce here a great change in the stresses, and conservative designers will modify the moments in Equations 10 to 13 according to their judgment.

A great change in the distribution of the moments is also caused by drops in the floors, which come now more and more in use, just at the columns, generally $\frac{4}{10}\,l$ square, and $\frac{1}{2}$ to $\frac{3}{4}$ t thick. The increase of stiffness around the columns increases the negative moments to a maximum of about $\frac{Wl}{15}$, and decreases the positive moments in the center to about $\frac{Wl}{60}$, instead of $\frac{Wl}{19.2}$ and $\frac{Wl}{32}$, as found before.

The flat slab floor with drop is a great improvement over a floor with uniform thickness, because the shearing stresses at the columns are decreased considerably thereby. The actual shear at the periphery of the cap is much larger than statics alone would lead us to adopt. The negative moment at the columns can be replaced by vertical forces acting in the opposite direction around the periphery of the cap, which new forces increase the shear on the loaded side and decrease it on the unloaded side of the columns. Assuming the side of the square

cap to be $\frac{2}{10}l$, the new shear on one side of the square $= \frac{Wl}{15} \div \frac{2}{10}l = \frac{W}{3}$, the shear from statics being $\frac{W}{4}$. **Mr. Mensch.**

C. A. P. TURNER, M. AM. SOC. C. E. (by letter).—The writer feels moved to contribute to the discussion of this subject, in which he has been greatly interested for the last sixteen years, because, in the course of his experience, acquired in the design and construction of from 1 000 to 2 000 structures of this type, varying in span from less than 12 ft. up to 50 ft. from center to center of columns, and built to carry loads from 50 lb. up to between 1 and 2 tons per sq. ft., he has become somewhat conversant with the commercial requirements of a working theory for such flat slab floors. These requirements may be stated in brief to be two: **Mr. Turner.**

First, it is necessary to know with certainty what test load can be guaranteed to be carried by the proposed slab without injury, and second, what limiting deflection can be guaranteed under test load. Such guaranty, both as to carrying capacity and deflection, is a common demand on the part of those furnishing the money for financing the construction of a proposed building or structure; and the conservative business man who advances the money, as the writer has found by experience, would usually like a bond, which may amount to anywhere from \$5 000 to \$100 000, to assure him that the structure when completed will come up to the guaranty.

Consequently, the responsible engineer must have an absolute knowledge of the deflection which his design will exhibit under load, and a feasible and practical method of determining its strength, such as will include a reliable estimate, within narrow limits, of the stresses that actually will occur in the steel under given loads.

In case a working theory can be developed along rational lines which will include these particulars, its accuracy as an application of the general theory of elastic materials can be readily checked by measurements on the behavior of the slab, to ascertain first, whether its actual deflections coincide with theoretically computed deflections, and second, whether the stresses occurring in the steel and in the concrete in each and every part of the slab agree with the results of mathematical theory. These checks and cross-checks would be such as would render the accuracy of the theory unassailable, or else would stamp it as a mere theoretical absurdity which in some of its fundamental assumptions does not correspond to and take correct account of the controlling factors of the design. Such a complete theory, further, must account for the fact that a thick slab will sustain around the cap higher shearing stresses per unit of cross-section than a thin slab, and must indicate what those stresses should be.

Mr. Turner.

In the experience above referred to, as already stated, the writer has been called on to design structures for loads varying from 50 lb. to between 1 and 2 tons per sq. ft. over full areas, and it has been necessary for him to know with a high degree of certainty what the deportment of slabs would be for short spans, and for long spans, for the limiting practical thickness, and for greater thicknesses, and to investigate carefully the maximum allowable percentage of steel for these various thicknesses in order that its resistance might be properly developed by the concrete and so determine a limit beyond which the addition of a greater percentage of steel is merely a waste of materials.

In the examination of this paper, none of these essential commercial requirements seems to have been mentioned, and no evidence whatever is offered in the form of experimental determinations to show that the results arrived at have any foundation other than that of the mere algebraic deductions which the author has based on certain assumptions.

These assumptions and deductions by Mr. Nichols appear to involve the most unique combination of multifarious absurdities imaginable from either a logical, practical, or theoretical standpoint. At the very outset he assumes the illogical proposition that the mechanics of a slab and the mechanics of a beam are identically the same, and then, lest this assumption appear doubtful, he makes a remarkable assertion in lieu of any proof of his statement in the following words:

"With whatever industry the powers of higher mathematics and the mysteries of Grashof's formulas and Poisson's ratio may be invoked, they cannot justify a result for the maximum stress in the steel smaller than the limiting value determined in this paper."

The writer has frequently found that mathematical formulas were mysterious and incomprehensible when he has tried to understand them before thoroughly digesting the notation and interpretation of the symbols used in the formulas and the definitions of the elementary technical terms in the discussion.

Now, in the case of Mr. Nichols, it would seem that the mysteries of Grashof are mysteries to him for this very reason, as he has apparently not mastered the notation and meaning of the symbols used by Grashof and other well-known writers on the mathematical theory of elasticity. In this the writer refers to Mr. Nichols' abuse of the greatly over-worked term, "moment," which is used to apply to so many different things. The kind of moment must be specifically understood, and defined, if it is intended to place different moments in an equation on a basis of equality or inequality. Eddy* clearly defines the external moment of forces acting on a slab or beam as apparent moments. Mr. Nichols designates the apparent moment as M. Eddy specifically defines true moment as the unit stress or summation of unit stresses,

* "Flat Plate Theory of Reinforced Concrete Floor Slabs," 1913.

Mr. Turner.

multiplied by their lever arm, which, in the case of a reinforced slab, would be the unit stress in the steel, multiplied by the area of the steel, multiplied by its effective lever arm. The form of a beam, which is narrow, limits the reinforcement to practically one-way reinforcement of no considerable width. Hence, in a beam, the apparent moment equals what Eddy would term the true moment. Or, this might be put in another way: The apparent moment equals the total effective internal moment, which in turn may be defined as equal to the true moment plus the moment of resistance of the equivalent lateral effects. By lateral effects the writer refers to the extent to which the stress in one system of reinforcement may offset or nullify that in another system of reinforcement by co-action or interaction between the two. Now, in a beam, due to its narrow width, the fact that all the bars run parallel to each other, these lateral effects reduce to zero. Hence for the beam, the apparent moment equals the true moment. In the slab, the apparent moment, M_x,* equals the effective internal moment M_I* which equals the true moment plus the summation of the lateral effects, and it is these lateral effects which Mr. Nichols inadvertently leaves out of consideration in his theory, thereby arriving at results differing from 100 to 200% from those which would be logically obtained by a proper consideration of the difference in the mechanics of the beam and the slab found in the forms which he has selected for discussion.

Perhaps the magnitude of this surprising error may be brought out better from consideration of the geometrical deformations than by what Mr. Nichols might term the mysteries of Poisson's ratio.

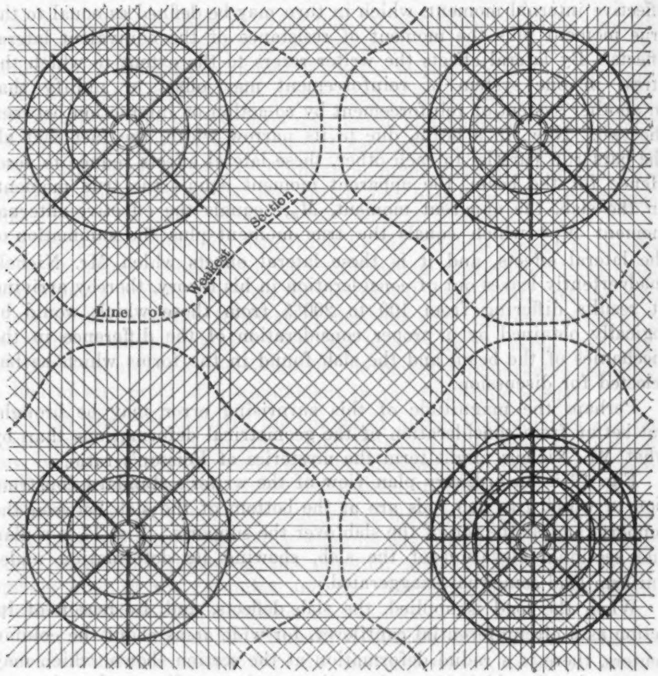
Fig. 16 is an ordinary plan view of the writer's standard construction in which the slab rods are at the bottom of the slab between the supports and at the top of the slab over the supports. Fig. 17 is a plan and sectional elevation of the slab, showing its deformation under uniform load somewhat exaggerated.

When the load is applied, the slab deflects, and, by reason of its action as a circumferential cantilever, for wide areas about the column centers, there is a radial deformation at the section shown, which may be termed ΔR, which is an elongation at the top fiber and a shortening or compression at the lower fiber. Now, the deflection being relatively small, this difference in length of R, measured on the neutral surface before and after bending, would be for approximate comparative purposes negligible. Hence it must be concluded from the necessary geometrical relations of the deformations that the total circumferential elongations on the upper surface are substantially $2 \pi \Delta R$, or, roughly, $6\frac{2}{7}$ times as great as the radial deformation.

Referring to Fig. 16, at the lower right corner, the formation of

*Turner's notation, "Concrete-Steel Construction," p. 26.

Mr. Turner. concentric octagonal polygons one within the other, has been accentuated by shade lines, and it becomes evident that the deformation of the slab is resisted directly by the radial resistances offered by the materials and also by the circumferential stresses induced in this series of concentric polygons, thus providing two kinds of support by which, under the principles of least work, the load may be carried,



SLAB RODS ACTING AS CIRCUMFERENTIAL FRAMES TO RESIST CIRCUMFERENTIAL DISTORTION

FIG. 16.

whereas in either the simple or continuous beam there is only one kind of support.

The determination of the load which would be carried by these respective resistances would (in accordance with the principles of least work) be in proportion to the rigidities of the resisting parts. However, it is unnecessary, for the purposes of this discussion, to go into the complete quantitative analysis of the efficiency of these two

Mr. Turner.

systems which appear in the cantilever portion of the slab, as the object is attained by pointing out the existence of these two systems by which the load may travel to its support, which constitutes a part of the fundamental difference between slab theory and beam theory, a difference which has been totally overlooked by Mr. Nichols.

The analogous action of the diagonal belts near the center of the panel brings out still more clearly the difference between slab theory

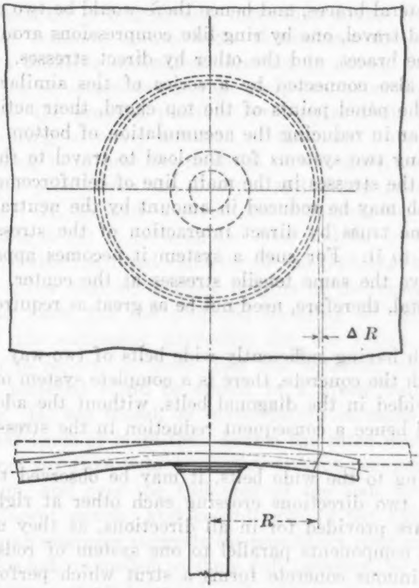


DIAGRAM ILLUSTRATING THE DEFORMATION OF A FLAT SLAB UNDER LOAD

FIG. 17.

and beam theory (or its equivalent), as applied to this part of the structure. This difference may be treated perhaps better in a rough, general manner by pointing out the analogy between the simple Pratt truss and the beam of homogeneous material in the manner by which the flange stresses increase toward the center. In a Pratt truss, for instance, having end panels with parallel chords, flange stresses are piled up or accumulated toward the center, according to the following laws:

The end chord stress is V_1 (tan. θ), the second panel is $(V_1 + V_2)$ (tan. θ), the stress in the third panel is $(V_1 + V_2 + V_3)$ (tan. θ), etc.

Mr. Turner.

Now, if in place of having a single truss to carry the load, there are two trusses intersecting at 90°, forming diagonals between supports, as in a slab, and there is in addition the condition that the trusses are stayed laterally at the panel points by heavy compression members, to similar posts in the truss normal to the first, there results the condition that the top chord stresses cannot accumulate toward the center in accordance with the law just mentioned, as the deformation in the direction of the line of the chord would cause a corresponding deformation in the lateral braces, and hence there would be two paths by which the load could travel, one by ring-like compressions around the squares formed by the braces, and the other by direct stresses. If the bottom chords were also connected by a series of ties similar to the struts assumed at the panel points of the top chord, their action would likewise be similar in reducing the accumulation of bottom chord stresses, thus furnishing two systems for the load to travel to the support. In like manner, the stresses in the main line of reinforcement at the center of the slab may be reduced in amount by the neutralization of the stresses in one truss by direct interaction of the stresses in another truss normal to it. For such a system it becomes apparent that one could not have the same tensile stresses at the center, and the cross-section of metal, therefore, need not be as great as required by the beam theory.

In the slab having sufficiently wide belts of two-way reinforcement, combined with the concrete, there is a complete system of lateral struts and ties provided in the diagonal belts, without the addition of more material, and hence a consequent reduction in the stresses involved in the steel.

In referring to the wide belts, it may be observed that with reinforcement in two directions crossing each other at right angles, tensile stresses are provided for in all directions, as they may be readily resolved into components parallel to one system of rods or the other, and the continuous concrete forms a strut which performs the functions of the suggested braces in the diagonal trusses assumed above for purposes of illustration.

A little consideration of these elementary geometrical relations will fully convince Mr. Nichols of the error of the treatment proposed by him. These actions and interactions between different belts of reinforcement have been treated in a highly scientific and exact manner by Dr. H. T. Eddy, in his recent work on the "Theory of the Flexure and Strength of Flat Concrete Floor Slabs," previously referred to.

This discussion would be incomplete did the writer not mention one of the many absurdities involved in Mr. Nichols' paper, from the practical standpoint; this is the remarkable assumption that any differences between the limiting stresses which Mr. Nichols has assumed must exist and those which actually occur in the practical building, must

Mr. Turner.

be due to the fact that a warehouse is never loaded to its capacity. To the man who has designed many such buildings and seen hundreds of them in use, such an assumption is a most interesting absurdity. In fact, the experienced constructor who takes pride in his work and guarantees it, never allows himself to entertain such a comfortable delusion. On the contrary, he is inclined to figure carefully on the story heights of the building which he is requested to construct for warehouse purposes, to determine whether there is room enough between the floor and the ceiling to store more than double the rated working capacity of common commodities throughout the floor. In all localities where building ordinances are not in force, the warehouseman who rents a structure for storage purposes naturally proposes to make it produce the greatest possible revenue, so that, if he takes a contract for storing so many hundred or thousand tons of sugar, his method of loading is to pile the material on the floor, commencing at the most distant point from the elevator and filling it up solid from floor to ceiling. If, in such a structure, the designer can calculate that it is possible to get in between the floor and the level of the ceiling, more than twice the rated capacity of the building, he is justified in suggesting either that these story heights be decreased or that the design working loads for the proposed building be increased. The writer makes this statement advisedly, for the reason that he has been called on to report, on more than one occasion, on the question of how much material could be stored in a given building without actual collapse. He has found, on investigation on more than a dozen warehouses, that loads of more than double the rated working capacity have been stored for from 3 to 4 months without permanent injury to the construction, and over the full area of the slab, or over a large number of adjacent panels; and accordingly, in the light of this experience, when an engineer suggests the possibility that such floors may never be overloaded, or loaded to their full capacity, he feels that the brilliant originator of such an idea should go out and visit a few ordinary warehouses and become somewhat familiar with commercial conditions before presenting so ludicrous a suggestion and one which is so likely to mislead the novice in design.

In the foregoing statement it should be borne in mind that the warehouses referred to as having been over-loaded were designed using a coefficient of bending of $\frac{WL}{50}$, which coefficient took into consideration the lateral effects of the four-way reinforcement. The unit stress used was 13 000 lb. per sq. in. for live load plus dead load. If the dead load is taken as one-third of the live load, and the live load is doubled, the working stress would be $1\frac{2}{3}$ times 13 000, or about 22 000 lb. per sq. in.—a very reasonable and moderate working stress for medium

Mr. Turner. steel having a yield-point value of at least 38 000 lb. per sq. in. If Mr. Nichols' discussion is correct, and this coefficient should be $\frac{WL}{19}$ instead of $\frac{WL}{50}$, it becomes evident that, instead of having a working stress of 22 000 lb., it would be something like 55 000 lb., or an amount greatly exceeding the yield-point value of the metal, and that, accordingly, if Mr. Nichols' method of computation is correct, the structure could not have sustained such a load without permanent deformation of the steel.

Looking at Mr. Nichols' paper in the light of an attempt to apply the impossible beam theory to the flat slab, it presents a practical absurdity in the manner in which it has been applied. Mr. Nichols proposes, quite properly, in considering his clear span, to take out a portion of the diameter of the cap, thus reducing the clear span, L. Now, for a continuous beam—and it is to be understood that he proposes to treat a continuous slab by his theory—the moments for an indefinite number of spans are $\frac{WL}{12}$ at the support, and $\frac{WL}{24}$ at the center. Splicing the rods over the head rapidly increases the metal, and any provision for shear which is essential for practical safety increases the metal at, and in the vicinity of, the supports. Such increase in metal increases the cantilever effect, thereby changing even for a continuous beam the relative moments that would be true or apply to a beam of uniform section, at the central portion. Why, then, should we assume for such a combination that a uniform stress in all the metal throughout should limit us to a value less than the mean of the moments at the support and at the middle? The proof of this proposition does not seem at all clear to the writer, from anything which is brought out in the paper, and, in fact, he is inclined to conclude that the results arrived at, even on beam theory, are due to a partial rather than a complete application of the principles of statics.

The various published tests of extensometer measurements on slab rods where four belts are used show that the stresses in the rods of the diagonal belts are smaller at the center of these belts than are those in the rods of the direct belts at the center of the span. If the beam theory governed in the distribution of the load on the respective belts, the stress in the steel should evidently be the same at the center of the diagonal belt as at the center of a direct belt, for a uniform load, and, when actual measurement shows a difference between the stresses measured and the beam theory, if assumed to apply, of nearly or approximately 70%, this difference is a fair measure of the lateral action in the rods of the diagonal belt which is not to be accounted for

by beam theory. This fact is brought out, not only in experiments carried on for the writer, but in those carried on by others.* Mr. Turner.

In considering these remarks the writer may add that while this criticism, though incomplete, may seem somewhat severe, it should be borne in mind that a complete and correct mathematical treatment of flat plate floors, from the theoretical standpoint, is a decidedly difficult matter. The writer has handled this problem successfully by a method of experimentally determined coefficients, which amounts to using the method of design by proportion utilized with such admirable results by trade guilds or associations of the Dark Ages in the production of monumental works in masonry and arches which stand unexcelled to-day. This method, however, is admittedly not as satisfactory as a complete rational analysis along conventional lines. Notwithstanding the fact that the writer has devoted more or less time for the past 16 years in attempting to develop such a theory, he has failed in doing so, and would probably be still at work on it had not the problem been solved satisfactorily by Dr. H. T. Eddy, Professor of Mathematics and Mechanics, Emeritus, of the University of Minnesota, and Dean of the Graduate School of Engineering of that institution. At the writer's request, Dr. Eddy has submitted a short discussion, in an endeavor to elucidate some of these simple relations which Mr. Nichols refers to vaguely as the mysteries of Grashof's formulas.

EDWARD GODFREY, M. AM. SOC. C. E. (by letter).—This paper is timely and important. In these days, when systems are springing up with rapid growth, based on nothing but tests that satisfy only the promoters of these systems, and builders who are anxious to put up structures at rock bottom prices, it is well to get at the base of things, and scrutinize the theoretical ground on which such systems purport to be based. Mr. Godfrey.

There was a time when theory did not bother the promoter. He considered his guaranty sufficient to hush all critics and to awe the boldest of them. Sundry wrecks have opened the eyes of the public and of engineers, and more or less research is being conducted to discover what is wrong.

In the early formulas for reinforced concrete beams, tension in the concrete was frankly used for its full value. The possibility of a crack just where the tension is the greatest did not seem to impress these theorists. Now, instead of using the tensile value of the concrete, the neutral axis of the beam is simply raised, and the allowed compression in the concrete is increased—just another way of doing the same thing—for tests to locate the neutral axis do not find it to be much, if any, above the center of the depth of the concrete beam. The theorists place the neutral axis very high, especially in a T-beam, which, of course, is a great advantage to the commercial builder of reinforced concrete.

* *Proceedings*, National Assoc. of Cement Users, Vol. 7, 1911.

Mr. Godfrey.

The loads specified for buildings are usually greater than the buildings will receive, so that if proper designs are made on this basis not a great deal of harm need result. Engineers, however, in building structures for rolling loads, should look deeper into the subject, if they want to build permanently.

In flat slabs supported on posts, the tensile strength of the concrete plays a still greater part in static tests than it does in beams, because the tension acts in all directions. Besides this, when an interior bay is tested, the aid rendered by the surrounding idle bays is great. In casting about for a plausible theoretical excuse, it was found that the neutral axis could not be raised a notch higher, hence, Poisson's ratio was drafted into service. When metal is in tension in one direction, it is extended in that direction, and contracts in a direction normal thereto. If now stress is applied normal to the first, it will tend to reverse the contraction in that direction and reduce the extension in the other direction. On the theory that stress is proportional to extension or so-called strain, it is argued that the tension in one direction, diminishes the effect of the tension normal thereto. This is the basis of Poisson's ratio. It is purely theoretical.

The writer has never found an engineer who was willing to use it in a hollow sphere under internal pressure, the most perfect case imaginable; in fact, spherical bottom tanks, instead of being made about half as thick as their tension would indicate, are commonly made twice as thick, a difference of about 400% from what this theory would sanction. So much for Poisson's ratio and confidence in the same, where the ratio is really applicable, if it has any application whatever in designing.

There are two experimental cases where Poisson's ratio might seem to have some meaning. These are doubtful, however. In a material that draws out like steel before ultimate failure, the ultimate strength is raised by stresses that prevent the "necking" of the specimen. This is exhibited in notched or grooved tests. This has nothing to do with the action of steel under stresses within the elastic limit. Glass, which is elastic up to failure, does not appear to exhibit this increased ultimate tensile strength, as will be shown later. The other case is where pressure is applied on a small area in the center of a broad flat stone or other similar substance. Much more pressure can be sustained than on a small cube, on account of the crowding of the material.

There is one place where Poisson's ratio has no application, and that is in the steel rods reinforcing a flat slab, the very place where it has been called in to bolster up false theory. The stress in steel rods reinforcing concrete can be only in the direction of the axis of the rods. It is astonishing what misapprehension exists regarding this subject. Some writers appear to think that Poisson's ratio

Mr. Godfrey.

is some mysterious thing by virtue of which a flat plate is supported by circular stresses, due to the dishing.

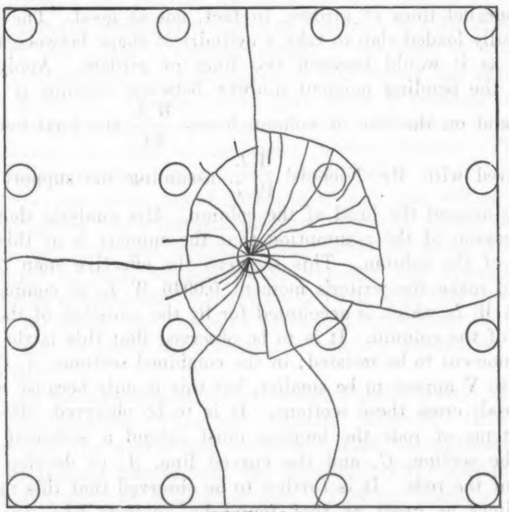
As stated by Mr. Nichols, the tests on flat slabs have been generally applied to single interior panels. Sometimes two or three interior panels are tested. There are no recorded tests of isolated panels with the load in the square enclosed by the columns, and none on exterior panels, except where the outer edge was supported by a girder on a wall. Furthermore, there are no tests of flat slab construction where load was placed on all panels across a building. This is, in fact, a critical loading. It is the criterion, which, if applied to the commercial systems, will show clearly their inadequacy. As the writer has pointed out,* a flat slab on rows of posts is no better than the same slab on parallel lines of girders, in fact, not as good. The tendency is for a fully loaded slab to take a cylindrical shape between two lines of posts, as it would between two lines of girders. Applying this criterion, the bending moment midway between columns is found to be $\frac{W\,L}{12}$, and on the line of column heads $\frac{W\,L}{24}$, the total being $\frac{W\,L}{8}$, as compared with Mr. Nichols' $\frac{W\,L}{10.7}$, assuming the support to be a rigid ring around the head of the column. His analysis shortens the span by reason of the assumption that the support is at this ring at the head of the column. This shortens the effective span to $0.87\ L$ and would make the writer's moment $0.0946\ W\ L$, as compared with his $0.0935\ W\ L$, which is accounted for by the omission of the load on the head of the column. It is to be observed that this is the smallest possible moment to be resisted; in the combined sections, *A*, *C*, and *B*, Cases II to V appear to be smaller, but this is only because more systems of rods cross these sections. It is to be observed, also, that in these systems of rods the lapping must extend a sufficient distance beyond the section, *C*, and the curved line, *A*, to develop the full strength of the rods. It is further to be observed that this moment is several times as great as that found by analysts who wrongly use Poisson's ratio.

The writer recently made some tests in an investigation of the stresses in flat plates supported on posts. The material used was glass. This material was chosen because it is elastic to the point of failure and will not sag or bag under a load. Test No. 1 (Fig. 18) shows how a central load in an interior panel acts. The glass was 24 in. square and about 0.09 in. thick. The sixteen circles represent wooden blocks, 1⅝ in. in diameter, which acted as posts. The load was balanced on a similar block at the center. The sheet of glass was held down by blocks over the sixteen posts. A load of 124⅝ lb. caused failure. Test No. 2 (Fig. 19) shows a similar piece of glass, similarly conditioned. On

* *Engineering News*, February 29th, 1912, p. 404.

Mr. Godfrey. the three blocks *A*, *B*, and *C*, there was placed a total load of 137 lb., when the glass broke. This sheet was not quite so thick. Evaluating the load to the same thickness as the other sheet, on the assumption that the strengths are as the square of the thickness, this load would be 157¼ lb. Instead of being three times what the interior bay carried, this is only about 1¼ times as much. This indicates that, even in a brittle substance which does not sag and act in suspension, the supporting power of an interior bay is no indication of the supporting power of a row of bays.

The general trend of the breaks in Test No. 2 in a direction parallel to a line through *A B C* indicates clearly the tendency of the slab to



TEST NO.1

FIG. 18.

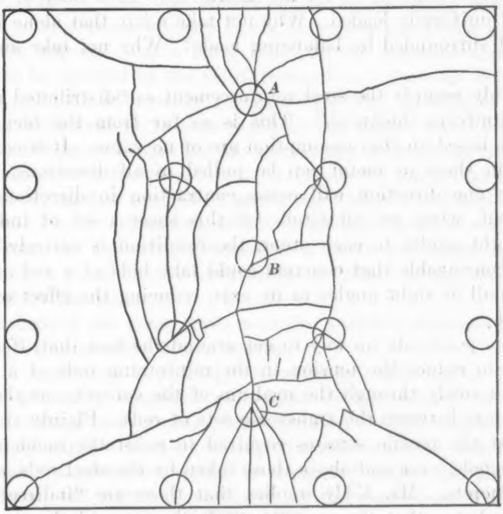
assume a cylindrical shape under loads in a row of panels. If any load would cause dishing in the middle of a panel, it would be a center load; but this dishing effect is practically nullified by consecutive panel loads, so that the mainstay of some theorists is destroyed.

It will be seen by the lines of the breaks in Test No. 1 that the glass broke in all directions in the center, where the load was applied and where the maximum intensity of stress evidently existed. Why did not conjugate stresses reduce the effect of stress at this point, as the theory of Poisson's ratio would indicate? Calculations of the modulus of rupture of some other tests of the series, which were smaller

Mr. Godfrey.

sheets supported on four posts, did not give results greater than for plain sheets in simple bending, in spite of the aid which conjugate stress is said to give.

An element of strength that is read into the slab itself (from the results of tests) is the bending resistance of the columns. The columns of a building are not calculated for bending, but, under a test load on the slab, they are only partly loaded, and can offer aid to the slab. This aid should not be counted on. It is dangerous to do so in a low structure or a bridge, where the full capacity of a column is apt to be reached.



TEST NO.2

FIG. 19.

There is no doubt of the fact that a large measure of the high showing of some tests on flat slabs of reinforced concrete is due to the tensile strength of the concrete. Tests on brittle material are therefore appropriate in an investigation of such flat slabs, especially where rolling loads or shocks are considered. Shocks in brittle material (concrete under tension) may cause rupture after months or years of use. What would happen in a proposed freight terminal, on flat slabs, when the shocks have cracked the concrete, and the steel receives its full stress, a long load down the middle of the slab causing it to act in bending as a cylinder, the calculated steel stress being about the elastic limit of the material?

Mr. Godfrey.

There is nothing in the flat slab to commend it for rolling loads.

It would be well if Mr. Eddy could be induced to make specific reply to the following criticisms of his theory. The writer has repeatedly made these criticisms publicly, but, so far as he knows, no reply to them has ever been made. Such responses as have been made have been evasive, and some have appeared to aim to throw a veil of obscure mathematics over the subject, to confuse and tire the reader.

Flat plates are being pushed to the front, more or less. The ordinary practising engineer has a right to know the mysteries of the subject, if there are any, in order to have them dissolved.

Mr. Eddy's analysis is for an interior bay of a floor of indefinite extent, all uniformly loaded. Why not take a bay that alone is loaded, and is not surrounded by balancing loads? Why not take an exterior bay?

Mr. Eddy regards the steel reinforcement as "distributed in a thin sheet of uniform thickness." This is so far from the fact that his deductions based on that assumption are of no value. It is conceivable that a thin sheet of metal can be pulled in all directions, and that tension in one direction will cause contraction in directions normal thereto; but, when we substitute for this sheet a set of independent rods at right angles to each other, the condition is entirely changed. It is not conceivable that concrete could take hold of a rod and by its adhesion pull at right angles to its axis, reducing the effect of tension in the rod.

There is positively no way to get around the fact that, if Poisson's ratio acts to reduce the tension in the reinforcing rods of a flat slab, it must act solely through the medium of the concrete, as there is no other medium between the transverse sets of rods. Plainly stated, this means that the tensile stresses required to resist the moments found by Mr. Nichols, over and above those taken by the steel rods, are taken by the concrete. Mr. Eddy implies that these are "indirect" tensile stresses, and says that they are "precisely the same kind as are always found to act in reinforced concrete beams." But building codes, and specifications, and practice, and even books, forbid reliance on these tensile stresses in resisting bending moments in beams. Why should there be this discrimination in favor of the flat slab? Strip it of its reliance on tensile stresses in the concrete, and what is left? No commercial flat-slab design would stand. Will Mr. Eddy, or any other champion of that system, attempt to deny this? If so, here is a concrete case: Given two rows of columns and a flat slab over their tops, acting as a viaduct to carry a railroad track midway between them: Suppose the jarring of the moving load has cracked the slab down the middle of the track or that contraction of the concrete has produced the same result. Now apply any commercial flat-slab theory with its

Mr. Godfrey.

"interior-bay-surrounded-by-balanced-loads" species of analysis. Next apply the theory of flexure to this slab—a simple beam supported along two lines (the rows of columns), balanced beyond these lines with dead load only. The bending moment along the crack is simple and perfectly definite. All the tension to resist this bending moment must be taken by the reinforcing steel. If the slab has been designed by any commercial flat-slab theory the stress in the steel will be found to be such that failure would be looked for. Until the flat-slab theorists and champions can deny this, it would be better not to use flat slabs for bridges or rolling loads.

Mr. Eddy.

H. T. EDDY,* ESQ. (by letter).—The fundamental erroneous assumption of this paper appears in its first sentence, which should state that statics imposes certain lower limits to the apparent applied forces that are to be resisted by the reinforcement in a flat slab floor.

It is assumed, apparently unconsciously, by the author, that this statement is equivalent to that made in the paper, for the entire discussion in the paper hinges on the correctness of equating the apparent applied moments to the moments of resistance of the actual tensions in steel reinforcement, a method of procedure which is entirely correct for a beam, but wholly misleading and entirely incorrect for a slab, as will appear from the following discussion of true and apparent bending moments in a plate or slab reinforced in either of the various ways considered in the paper.

The source of the error appears in the author's statement regarding Grashof's formulas and Poisson's ratio.

Such a statement is entirely unwarranted, and not in accordance with the generally accepted principles of the theory of elasticity. There is nothing mysterious about Grashof's fundamental formulas. They simply express mathematically the experimental fact that, when a solid piece of elastic material is elongated under the action of an applied force, the material undergoes at the same time a lateral contraction, the amount of which is expressed by Poisson's ratio. That fact, the writer assumes, would not be denied by the author.

The fundamental equations of extensional stress and strain in thin flat plates and slabs, as given by Grashof,† and accepted by all authorities on the theory of elasticity since then, may be written in the forms:

$$\left.\begin{aligned} f_{s_1} &= Ee_1 = p_1 - Kp_2 \\ f_{s_2} &= Ee_2 = p_2 - Kp_1 \\ \text{or,}\quad (1 - K^2)\,p_1 &= E\,(e_1 + Ke_2) \\ (1 - K^2)\,p_2 &= E\,(e_2 + Ke_1) \end{aligned}\right\} \qquad (1)$$

* Professor of Mathematics and Mechanics, Emeritus, College of Engineering, University of Minnesota.

† E. Grashof. "Theorie der Elasticität und Festigkeit." p. 352, 2d Edition, Berlin. 1878.

Mr. Eddy. in which p_1 and p_2 are the external applied or apparent stresses per unit of area of cross-section of the plate or reinforcement of the slab which act parallel to the axes of x and y, respectively, if these latter lie in the neutral plane of the slab, and are parallel to the edge of the panel; and e_1 and e_2 are extensometer elongations of the plate or slab reinforcement per unit of length parallel to x and y, respectively, while f_{s_1} and f_{s_2} are the corresponding true stresses in the steel. E is Young's modulus and K is Poisson's ratio of lateral contraction to linear elongation. Any piece of material which is subjected to stress, and is of such shape that more than one of its dimensions is considerable, must have its stresses and strains considered with reference to lateral contraction. This is the case in plates and slabs, as it is not the case in rods and beams.

In the foregoing equations, Ee_1 and Ee_2 are the true stresses per square inch of section of reinforcement acting along the lines parallel to x and y, respectively, whatever p_1 and p_2 may be. These latter are the cause of true stresses, but are not themselves the values of the true stresses, as in case of rods, etc., where one dimension only is large. These equations show that the elongation, e_1, in the direction of x and y, is not dependent alone on the tension, p_1, applied in that direction, for it is diminished by any tension, $+ p_2$, acting along y, but is increased by any compression, $- p_2$, along y. It appears that any tension, $+ p_2$, assists the piece in resisting the elongation along x and makes it able to endure safely a larger applied stress, p_1, with the same degrees of safety, that is, with the same percentage of elongation or true stress; but it is equally true that any compression, $- p_2$, reduces the safe value of p_1 which may be applied to it. These principles are not in accordance with those which hold for rods and beams, the lateral dimensions of which are small compared with their lengths. This divergence between the true stresses, as shown by actual deformations, and the apparent applied stresses, is a fruitful source of error in the attempted computation of slabs. Equations (1) in their present form apply to simple extensional or compressive stresses and strains, but may be extended to apply to bending of slabs, in the following manner:

Take A as the cross-section of the reinforcement per unit of width of slab when the actual reinforcement is regarded as distributed in a thin sheet of uniform thickness, and let jd be the vertical distance from the center of the reinforcement to the center of the compressional resistance of the concrete regarded as a fraction, j, or, d, the distance from the center of the steel to the top of the slab. Then

$$A p_1 \; jd = m_1 \text{ and } A p_2 \; jd = m_2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

are the apparent bending moments of the applied apparent stresses, p_1 and p_2, per unit of width of slab, tending, when positive, to cause

lines which, before, bending, are straight and parallel to x and y, respectively, to become convex downward. Mr. Eddy.

Again, $Ee_1\, Ajd = m_1$ and $Ee_2\, Ajd = m_2$ ....(3)

are the true bending moments of the actual resistance stresses in the reinforcement per unit of width of slab, as shown by the extensometer strains in the steel parallel to the axes of x and y, respectively.

Multiply Equations (1) through by Ajd and substitute the values given in Equations (2) and (3), from which is obtained the following relations between the true and apparent bending moments in the slab:

$$\left.\begin{aligned} m_1 &= \mathrm{m}_1 - K\mathrm{m}_2 \\ m_2 &= \mathrm{m}_2 - K\mathrm{m} \\ (1 - K^2)\,\mathrm{m}_1 &= m_1 + Km_2 \\ (1 - K^2)\,\mathrm{m}_2 &= m_2 + Km_1 \end{aligned}\right\} \text{..................(4)}$$

These equations bring out in a striking manner the essential divergence of the correct theory of slab action from that of beam action in which latter case there are the well-known equations, $m_1 = \mathrm{m}_1$ and $m_2 = \mathrm{m}_2$, that is, the moment of the applied forces is equal to the moment of the internal resistance, which is not true of slabs.

All attempts to base the computation of bent slabs on beam action are necessarily erroneous, for it is wholly inapplicable and misleading. Deflections and stresses in slabs cannot be computed correctly by any form of simple or compound beam theory.

Equation (4) shows:

1st.—That at points where the apparent moments, m_1 and m_2, are of the same sign (as, for example, in the convex part of the slab near the columns and also near the center of the panel), the true bending moments, m_1 and m_2, which determine the true stresses in the reinforcement, are less than the apparent bending moments in which the latter have been ordinarily assumed (according to the beam theory) to determine these stresses.

2d.—That the compressive stresses in the concrete around the column cap are determined on the same principles, and are consequently reduced in accordance with the value of K by a considerable percentage below values corresponding to m_1 and m_2 of the beam theory.

3d.—That the points where m_1 and m_2 have different signs, as they have, for example, in the middle part of the space directly (not diagonally), between the column heads, the values of the true bending moments are larger than the apparent moments as found by the beam theory.

4th.—One of the results of this is the fact, which is also completely confirmed by extensometer tests as well as theoretically, that the greatest actual extensions and true unit stresses in the reinforcement of slabs, as ordinarily arranged, occur at the mid points of the rein-

Mr. Eddy. forcing rods which run directly between the column heads parallel to the sides of the panel and do not occur in the diagonals, at the center of the panel where m_1 and m_2 have their greatest values. Further, with suitable laps, the true stresses in the reinforcement are not so large over the columns as at the points just indicated. Neither of these conclusions is in accordance with the beam theory, as implied in ordinary formulas adopted for practice.

5th.—Any requirement or statement as to the bending moments at any point of a slab must state which kind of bending moments is called for, the true bending moments or the apparent moments, with the understanding that the true bending moments only are to be used in determining cross-sections of steel. Any requirement seeking to proportion the cross-sections of steel to apparent stresses and moments is incorrect.

It is evident from Equations (1) and (4) that Poisson's ratio, K, plays an important rôle in the theory of flat slabs and plates. Few attempts have been made to determine K by directly measuring the amount of the lateral contraction accompanying the elongation of test specimens, and, were such measurements made, the relative dimensions of the cross-section of the specimen would need to be considered as affecting in a very complicated way the true value of K to be derived from observation. Reliable determinations of K usually depend on observations of Young's modulus of elasticity, E, and the shearing modulus of elasticity, F.

It is proven, in the general theory of the deformation of isotropic elastic solids, that all the elastic properties of any such solid are determined without excess or defect by its values of E and F, and that Poisson's ratio is a function of E and F expressed by the equation:*

$$K + 1 = \frac{1}{2} \frac{E}{F} \qquad \text{(5)}$$

There is evidence to show that, for concrete, K is approximately 0.1.† For steel, it is known that $K = 0.3$, nearly.

Now, it is evident that a horizontal slab of reinforced concrete, in which the reinforcement consists of rods, differs from one in which the reinforcement is considered to be a simple uniform sheet of metal, in this, that the former has much less shearing rigidity in resisting horizontal forces than the latter, for in it all stresses transmitted from one band or belt of rods to any other belt crossing it are transmitted through concrete only, as is not the case if the reinforcement consists of a continuous sheet. It is evident, therefore, that the value of K which

*Merriman's "Mechanics of Materials," 10th Edition. 1911, Equation (181), p. 466.

†Turneaure and Maurer's "Reinforced Concrete Construction," 2d Edition, 1912, p. 272 b.

must be used in applying the foregoing equations to reinforced concrete slabs must exceed 0.3, the value required in case the reinforcement is a sheet of steel. Mr. Eddy.

It may be objected that the transmission of longitudinal stress from the rods of one belt laterally to those of any belt crossing it involves secondary stresses in the concrete which we are not justified in assuming it to be capable of resisting. However, such objection is untenable, as is evident from the phenomena exhibited in the flexure of reinforced concrete beams, which show that the concrete retains its shearing bond with the reinforcement and transmits horizontal shears as secondary stresses to the neutral surface and to the concrete in the zone of compression long after it has passed the limit of its tensile strength. Were it otherwise, reinforced concrete beams would be of no practical utility. Action of this kind likewise occurs in the reinforced concrete slab where the bond between the concrete and the belts of rods embedded in it bring them into co-action with each other in a manner like the co-action of the parts of a reinforced concrete beam.

This analysis of the conditions affecting the value of K for a reinforced flat slab differs radically from assuming at random that because $K = 0.3$ for steel alone and $K = 0.1$ (possibly) for concrete alone, that therefore some intermediate value of K may be correct for these two materials combined in a slab. Such an assumption is merely a blind guess, and has no rational basis.

As already partly stated, the view here put forth is this: Since, in any homogeneous, isotropic, elastic material, the experimental values of E and F perfectly define all its elastic properties, and since we are evidently at liberty to assume our flat slab as sufficiently fine grained in its structure to act nearly like a slab constructed of some sort of homogeneous materials, it will be possible to determine certain mean values of E and F which will define its elastic properties. It is, moreover, evident that in a slab, where two kinds of elastic solids are combined as they are here, the mean value of F for the combination is affected much more by the concrete than is E, which latter may be taken as that applying to the steel alone, and, consequently, as unchanged by the combination. It is otherwise, however, with F, because the arrangement of the combination is such as to require the assumption of a value of F lying somewhere between that for steel and that for concrete. As the latter value is much less than the former, the mean value of F is smaller than for steel alone.

This reasoning and other independent theoretical and kinematical considerations have led to the same conclusion, namely, that the correct value of K for the slab is larger than 0.3.

Mr. Eddy.

Assuming $E = 30\,000\,000$, we may compute the corresponding values of K and F from Equation (5) as follows:

$$\text{If } K = 0.1,\ F = 13\,600\,000.$$
$$\text{If } K = 0.3,\ F = 11\,600\,000.$$
$$\text{If } K = 0.5,\ F = 10\,000\,000.$$

If a perfectly complete and accurate mathematical theory of the flat slab were at our disposal, we might consider every experimental test of the deflection of such a slab, and every extensometer measurement of its reinforcing rods as an experiment for determining the numerical value of K, as deflections and extensions would then all be known functions of K. Having brought such a rational theory to a somewhat satisfactory degree of perfection, the writer finds that, in the light of all known tests of cantilever flat slabs, of the standard mushroom type, with four-way reinforcement covering the entire slab area, the value that best satisfies all conditions is:

$$K = 0.5 \quad \text{..........................(6)}$$

It is possible that this value of the constant, K, for slabs may need some slight modifications hereafter, but for the present this may be regarded as substantially correct for such slabs. It may be found necessary to assume a somewhat different value for other forms of structures, as, for example, beam and girder construction. That, however, must be determined later. Moreover, it must be said that this value of K applies to tests of slabs from 2 to 4 months old, and under loads which have been applied to such relatively soft concrete as this for a period of usually not longer than 1 or 2 days, and of an intensity such as to cause a maximum stress in the steel of from 10 000 to 20 000 lb. per sq. in. Less loads on better cured concrete, or longer time under load, may possibly show some deviation from this value of K.

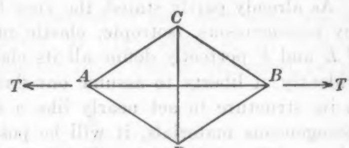


FIG. 20.

How important a factor K is in slab theory is evident on considering Equations (4) which show that in a square panel, uniformly loaded, the true moments, as shown by the elongations of the reinforcing rods at the center of the panel, and over the centers of the columns, are only one-half the corresponding apparent moments derived from considering the moments required to hold the applied forces in equilibrium, this being on the assumption, of course, that $K = 0.5$.

A further and somewhat more elementary consideration of the questions at issue may put the matter in clearer light, and facilitate the discussion.

Let *A B C D*, Fig. 20, be any stiff diamond frame (which for the present will be assumed to be made of steel) such that the joints at the corners are of unknown stiffness. The sides, too, are of unknown inclination and cross-section. In addition, let *A B* be a tension member of known cross-section, and *C B* be a compression member, also of known cross-section. Mr. Eddy.

If known tensions, *T*, were applied at the corners, *A* and *B*, it would be impossible, under the circumstances, to determine by the laws of statics alone what stress would occur in *A B* or *C D*. So long as the relative dimensions, inclination, cross-sections, and other structural properties of the parts of the diamond frame are unknown or hidden, it is absolutely impossible by mere statics to find out how much of the applied forces, *T*, are resisted by the tie, *A B*, or the strut, *C D*, and any attempt to correlate these forces with the stresses on such a basis would be entirely without foundation.

In default of any structural data respecting the diamond frame, the only recourse would be to experiment on the properties of the proposed arrangement. Now, this is precisely what occurs in the reinforced concrete slab. The known members, *A B* and *C D*, represent parts of the reinforcement, and the diamond frame acts like the matrix of concrete in which they are embedded. The properties of the matrix in combination with the reinforcement are not readily amenable to analysis, and the mechanics of its action is largely hidden from view, but, whatever it may be, it is not difficult to discover the general nature of its action.

What this is, is evident from a careful consideration of what goes on in a simple concrete beam reinforced with steel rods near its lower surface. Though it is not permissible to consider direct tension in the concrete as an element of strength, it is not only permissible but necessary to consider the bond value of the shearing stresses in the concrete enclosing the rods, as well as the horizontal shears in the beam. These last, in fact, are absolutely essential to the action of the beam in order to transmit the tension in the steel at the bottom to a parallel position vertically above it where it can be neutralized and held in equilibrium by the compressions of the concrete in the upper part of the beam. However, such shears, vertical and horizontal, as are here admitted to exist, constitute a state of stress in the concrete in which indirect tensions and compressions exist on planes inclined to the planes of shear. It thus appears that, though direct tension may not be relied on as an element of strength in concrete, the same is not true of indirect tension, which is not only admissible but always necessary for beam action.

Now, in a slab, just such indirect tensions and compressions occur in the distribution of stresses laterally in the concrete as occur in the beam in distributing them vertically, and it is by means of them that

Mr. Eddy. horizontal stresses in one direction in the slab are brought into co-action laterally with stresses in other horizontal directions without producing what would otherwise be their full effect on the steel. In other words, the matrix represented by the diamond frame, $A\ B\ C\ D$, acts in such a way as to resist part of the applied force, T, without itself undergoing anything except permissible indirect stresses such as occur in beam action. In a way, the horizontal dimensions of a slab play a part to reduce both direct and indirect stresses in a manner analogous to the reduction of such stresses in a beam by reason of its vertical dimensions.

Here is an element of strength and resistance in the slab heretofore entirely disregarded, but one which must be considered just as unavoidably as in reinforced beams. If it cannot be neglected in one case it cannot in the other, in any rational analysis of the factors that must be taken account of in the behavior of slabs when considering their strength and resistance.

The final effect of this analysis is to show that, although direct tension in concrete is not admissible, and not to be counted on as an element of strength, the unavoidable indirect tensions in the concrete are such as to play a most important part in resisting the applied forces; and that, consequently, the reinforcement is called on to resist only part of the tension which is caused by the applied forces. The reinforcement may be regarded as in a sense protected by the concrete from much of the tension it would be compelled to sustain were the concrete not thus co-acting with it. The divergence of results reached by Mr. Nichols and the writer is largely due to this consideration.

The conclusions arrived at in this discussion which have not been heretofore explicitly summarized may be stated as follows:

1st.—The attempt to derive the stresses in multiple-way reinforcement of a slab from the equations of statics is futile, and cannot possibly lead to correct results.

2d.—This is not merely because the applied forces in a given direction are only partly borne by reinforcement in that direction and partly by all the lateral reinforcement there may be in other directions (even by that at right angles), but because the bond of the concrete and steel necessarily calls indirect stresses into action in the concrete, which help resist the applied forces, such indirect stresses being of precisely the same kind as are always found to act in reinforced concrete beams.

3d.—This method of co-action of steel and concrete gives no warrant whatever for the assumption that it is ever safe or proper to rely on unreinforced concrete for resistance to direct tension.

4th.—In case of one-way reinforcement in the direction of the applied forces, the concrete is an element of strength, by reason of its compressive resistance at right angles to this direction, even though

the concrete may be so full of hair cracks at right angles to the applied tensions as to afford practically no resistance to them. Mr. Eddy.

5th.—Any area of concrete not situated so as to be tributary to reinforcement, so that the steel can co-act efficiently with it, cannot be subjected to tensile stresses beyond certain limiting intensities without cracking.

6th.—Every safe slab must have reinforcement on its face at or near where tensile forces act, in order to co-act with the concrete in resisting them without prohibitive direct tensions in the concrete.

An instance of this co-action too frequently overlooked is that of the steel over the column heads, which, if sufficiently stiff, prevent prohibitive tensions in the concrete across the top of the slab over the side belts of the panels, even at the considerable distances of the mid-span.

7th.—Although, in its total effect on reinforced concrete, the action of indirect stresses, as just developed, separates the resistance to the applied tension into two parts, one resisted by the reinforcement, and the other by the concrete, it requires the presence of the steel in order to develop any tensile resistance in the concrete, and when the steel is not present, no direct tensile resistance whatever is to be ascribed to the concrete. This is believed to be in entire accordance with good practice, and with the consensus of opinion among responsible engineers, as well as in accordance with most city ordinances on the subject, and is a principle that should be adhered to.

Consider the case of a rectangular slab supported at its four edges by a frame consisting of more or less flexible girders, resting on columns at its corners, and reinforced by two-way diagonal rods. Such a slab is in the condition of that central portion of an ordinary slab with four-way reinforcement which lies between the lines of inflection.

When uniformly or centrally loaded, its concave shape is really the combined effect of two hollow troughs superimposed on each other, each extending across the slab at right angles to the other and parallel to the sides, giving a sort of groined effect upside down. Now, it is clear that a plate or slab when bent in this shape is much stiffer than the same slab bent into the shape of a single trough, like a simple beam across one way of the slab, because each of the troughs exerts a resistance to the bending that produces the other.

For the moment, this increase of stiffness due to the resistance arising from the interaction of these two troughs may be designated as slab action. What circumstances tend to make the bending such that one of these troughs is intensified at the expense of the other? It would be agreed that a slab or plate resting on two rigid walls along its opposite sides would have a single cylindrical trough between the walls, of a shape like that of the deflection curve of a simple

Mr. Eddy. beam. If, now, in place of each wall, each of two edges should be supported on a row of columns, the distance from center to center of which is small compared with the width between the rows, this would accomplish nearly the same thing.

It thus appears that in any single panel having a length which is great compared with its width, the trough and deflection lying between the ends in the longer span would be much larger than that between the sides, and this trough that extends across the width of the slab tends much more strongly than the other to gain the upper hand and extinguish the one opposed to it. This larger trough is reduced somewhat, and the deflection diminished somewhat below what would occur in a simple beam the long way of the slab, by the trough extending lengthwise of the slab, and this crosswise trough will be obliterated to a very large extent.

Now, it is not necessary to state that the railroad bridge proposed by Mr. Godfrey has its long spans lengthwise and not crosswise of the track, but, as has just been shown, the principal bending due to the long spans tends strongly to obliterate the crosswise bending; and yet Mr. Godfrey states that the slab bridge will necessarily crack lengthwise of the track. The writer has just shown that this could not possibly occur until long after the slab had given way by yielding following cracks across the width of the slab, such as would be impossible in a properly designed bridge.

Mr. Godfrey is misleading in his statement of the writer's position respecting direct and indirect tensile stresses in concrete and their action, and does not understand that the writer is as strongly opposed to relying on direct tensile stresses in concrete as he is. The writer, however, insists on the reliability of the shearing bond between a reinforcing rod and the concrete in which it is embedded; and assumes, further, that Mr. Godfrey agrees with him in this.

Now, a shear in the concrete on any plane lengthwise of the rod, according to the fundamental principles of stress, implies an indirect tensile stress in the concrete in a direction at 45° with the axis of the rod. It is the action of the bond shear in producing this indirect tensile stress in a diagonal direction that the writer has been considering, and not, as Mr. Godfrey states, a tensile stress in the concrete at right angles to the rod. In nothing that the writer has stated is there justification for assuming any tensile stress in the concrete perpendicular to the rod. What he has assumed, however, is: that the same concrete that has shear bond with one set of rods has at the same time shear bond with another set of rods perpendicular to the first set, and that the indirect diagonal tensile stresses due to extension in the first set of rods would tend to produce compressions in the second set, which perhaps may not become effective in the second set, actually to produce any compression in it, but does become effec-

Mr. Eddy.

tive at once when the second is subjected to elongation, and will inevitably act to resist elongation of the second set. It thus comes about that in a slab with multiple-way reinforcement, where all the rods in a given area are in tension, they interact by means of the bond shear of the concrete in a manner that insures results of the same nature as those found in a solid plate. It is also found that, even in cases where direct tensile stresses in the concrete have caused cracks at right angles to the rods, which have certainly destroyed its tensile strength, the shear bond is still mostly intact and the action previously described is not diminished to such an extent by this fact as to prevent it from reducing materially the stresses which must otherwise exist in the steel. It thus appears that diagonal indirect tensile stresses, which have a component parallel to direct tensile stresses in the steel, carry part of the steel stresses, without thereby calling on the concrete for any direct assistance, and this occurs even in cases where it is known that direct tensions cannot exist because of visible cracks.

Mr. Godfrey's discussion, furthermore, has raised an important question which may be put in this form: In case of unbalanced moments at a column due to unbalanced load in the panels, how much of the unbalanced moment is resisted by the column, and how much by the action of the slab itself?

In the mushroom system, the connection between slab and column is of unusual rigidity, and therefore likely to cause greater bending moments in the columns than would occur in other systems not joined so rigidly.

Experimental data secured in the test of the Northwestern Glass Company Building, by Mr. F. R. McMillan, permit a determination of the actual bending moment in Column No. 45, at the point where the gauge lines were observed. How much the total unbalanced moment that must be resisted by column and slab may be, is a question complicated by the amount of restraint exerted by both column and slab, and not here investigated; but, whatever it may be, it is evident that only so much of it as remains after subtracting that carried by the column must be resisted by stresses in the slab, rods, etc., a result which has to do with the actual stresses in those rods.

The actual bending moment in Column No. 45 of the Glass Company building test, under unbalanced load No. 7 of 185 000 lb., on Panel D, may be computed as follows:

The deformation data of Column No. 45 are given on page 29, of Mr. McMillan's report, and the location of the gauge lines on his Drawing No. 3.

The observed data are very conflicting because of lack of sufficient checking of zero readings, changes of temperature, and initial shrink-

Mr. Eddy. age, but prolonged analysis has shown how they may be brought into harmonious relation with each other and the applied loads by reasonable adjustment of the observations, which meets the approval of Mr. McMillan.

It is clear that the readings under Load 1, having a mean intensity of 112 lb. per sq. ft., and distributed practically uniformly on the four panels about Column No. 45, should have produced the same reading at all six of the gauge lines and compression at each of them instead of the large tensions shown at four of them with small compressions at the other two. Leaving out the deflections under Load 1, on the supposition that irregularities have been largely eliminated by its application, take the additional deflections caused by additional loads as shown in Table 1, in which the increment of the deformation above Load 1 is tabulated with the corresponding increment or addition to Load 1 that produced it.

TABLE 1.—ADJUSTED DEFORMATIONS, TAKING THE SLAB POSITION UNDER LOAD NO. 1 AS ZERO.

Deformations in hundred-thousandths of an inch, and loads in pounds.

Load.	*A*	*B*	*C*	*C'*	*D*	*E*	*E'*	*F*	Average.	Load, in pounds per square foot.
1-1	0	0	0	+ 30	0	*E*	+ 15	0	......	112
2-1	− 24	− 18	− 46	− 16	− 20	− 44	− 29	− 28	− 22.5	138
3-1	− 36	− 20	− 64	− 34	− 32	− 62	− 47	− 40	− 35	238
3-1	− 38	− 38	− 74	− 44	− 40	− 76	− 61	− 56	− 46	238
4-1	− 38	− 36	− 66	− 36	− 34	− 76	− 61	− 52	− 48	238
5-1	− 58	− 54	− 58	− 28	− 30	− 42	− 27	− 34	− 38.5	184
5-1	− 66	− 48	− 56	− 26	− 26	− 30	− 15	− 26	− 34.5	184
7-1	− 106	− 92	− 46	− 16	− 18	− 48	− 63	+ 40	− 20	62
7-1	− 110	− 96	− 48	− 18	− 22	+ 68	+ 83	+ 52	− 19	62
9-1	− 30	− 12	− 18	+ 12	+ 2	0	+ 15	− 6	......	......
9-1	− 42	− 38	− 54	− 24	− 22	− 26	− 11	− 32	......	......

It will be seen that an addition of 238 lb. per sq. ft. on each of the panels produces a mean deflection of about 0.0004 in. in a gauge line 8 in. long. On this basis the mean deflection due to 100 lb. per sq. ft. uniformly distributed would be about 0.00017 in. per 8 in. This is confirmed by the following independent computation of the load under a given deformation. The load actually carried by the column under a uniform load of 100 lb. per sq. ft. is $16 \times 17 \times 100 = 27\,200$ lb., which may also be computed as follows, provided the stresses are so low that the entire resistance of the column may be assumed to be carried by the vertical steel and the core, and none of it by the spiral and shell:

$$W = e(A_s E_s + A_c E_c) \dots\dots\dots\dots\dots\dots (a)$$

in which e is the unit deformation, and the areas of cross-section of steel and concrete in the column are $A_s = 6$ sq. in., $A_c = 566$ sq. in., there being six $1\frac{1}{8}$-in. vertical rods and a 27-in. core. Mr. Eddy.

$$27\,200 = e(6 \times 30\,000\,000 + 566 \times 2\,000\,000).$$

Therefore $8e = 0.000166$ in., which agrees well with the observed deformation per 100 lb.

In Table 1, it is evident that the readings in Column C disagree greatly with those in Column D, for which no reason can be assigned except some disturbance of the zero form in which all the readings in Column C are taken. In Column C' these readings have been corrected arbitrarily by 30 units, which brings them into good agreement with Column D, and the other columns. Similarly, Column E' has been obtained from Column E by an arbitrary correction of 15 units throughout.

TABLE 2.—ADJUSTED DEFORMATIONS MEASURED FROM THE ORIGINAL ZERO.

Deformations in hundred-thousandths of an inch, and loads in pounds.

Load.	A	B	C'	D	E'	F	Average.	Load, in pounds per square foot.	Live load on column.
1........	− 19	− 19	− 19	− 19	− 19	− 19	− 19	112	30 300
2........	− 43	− 37	− 35	− 39	− 48	− 47	− 41.5	250	67 900
3........	− 55	− 39	− 53	− 51	− 66	− 59	− 54	350	95 000
3........	− 57	− 57	− 63	− 59	− 80	− 77	− 65	350	95 000
4........	− 57	− 55	− 55	− 53	− 81	− 73	− 62	350	95 000
5........	− 77	− 73	− 47	− 59	− 48	− 55	− 57.5	296	80 000
5........	− 85	− 67	− 45	− 45	− 36	− 47	− 57.5	296	80 000
7........	− 125	− 113	− 35	− 37	+ 44	+ 21	− 41	174	47 400
7........	− 139	− 115	− 37	− 41	+ 64	+ 33	39	174	47 400
9........	− 49	− 31	− 54	− 56	+ 25	+ 2	− 27	....	3 500
9........	− 61	− 57	− 56	− 60	+ 45	+ 14	44	....	3 500

Now, the compression due to Load 1 of 112 lb. per sq. ft. is about 19 units, as already seen. Assuming this to be correct, we have increased each compression in Table 1 by 19 units, and so obtained in Table 2 a set of adjusted values of the deflection reading from the original zero. These values are in sufficiently good agreement with each other and the loadings to enable us to base computations on them which it is thought will give results that are approximately correct.

Now, under Load 7 the mean compressions at AB may be taken at 123 units and the mean tensions at 41 units, so that the deformation at the extreme fiber due to bending may be taken as $\pm \frac{1}{2}(123 + 41) = \pm 82$ units per 8 in. long. In order to compute the bending moment producing this deformation, we write:

$$M = \frac{(f_s I_s + f_c I_c)}{r}$$

Mr. Eddy. in which the moments of inertia of steel and concrete may be taken as $I_s = \frac{A_s r^2}{2}$ and $I_c = \frac{A_c r^2}{4}$ by assuming the steel to form a thin cylinder on the surface of the core or radius, $r = 13.5$ in. and $e = \frac{82}{800\,000} = 0.0001025$ in. This equation may be written as follows:

$$M = r\,e\,\frac{(2\,A_s E_s + A_c\,E_c)}{4} \quad \text{.....................(b)}$$

$$M = 516\,000 \text{ in-lb., or } 43\,000 \text{ ft-lb.}$$

This is the bending moment at a point of the column at the mean level of the gauge lines, 6 ft. 8 in. = 80 in. above the floor.

Assuming that the bending moment increases uniformly from the point of inflection of the column to the bottom of the cap, which may be taken to be about 7 ft. 9 in. = 93 in. above the floor, we proceed to compute the moment at this point at the bottom of the cap.

Where the point of inflection of the column will be, depends on the kind of restraint at the bottom of the column. Assume the most unfavorable case, namely, that the column is fixed in position and direction at the bottom. The point of inflection where the moment vanishes will then be taken at 31 in. above the floor, and $M_1 = 54\,400$ ft-lb. Comparing this with $WL = 185\,000 \times 17 = 3\,145\,000$ ft-lb., we have $M_1 = \frac{WL}{58}$.

The dead load on the column was estimated at the time of the test to be 255 000 lb. This, added to $\frac{185\,000}{4} = 46\,250$ lb., makes a total column load of 301 250 lb. under Load 7, and the dead load, which is equivalent to a deformation of nearly 190 units in 8 in., or a stress in the steel of

$$f = \frac{190 \times 30\,000\,000}{8 \times 100\,000} = 7\,125 \text{ lb. per sq. in.}$$

Now, the steel stress at the outer fiber due to the deformation of 82 units previously obtained is

$$\pm \frac{82 \times 30\,000\,000}{800\,000} = \pm\, 3\,075.$$

The greatest and least compression stresses in the steel at the level of the gauge lines 80 in. above the floor are 10 200 and 4 050 lb. per sq. in., respectively, and the concrete stresses are one-fifteenth as much, namely, about 700 and 200 lb. per sq. in.; but, at the level of the bottom of the cap, the stress due to the unbalanced moment may be as much as $\pm\, 3\,075 \times \frac{62}{49} = \pm\, 3\,891$ lb. per sq. in. at the extreme fiber.

This, combined with the mean compression of 7 125 lb. per sq. in. due to the combined live and dead loads, makes the greatest and least steel compressions 11 016 and 3 234 lb. per sq. in., respectively, and the concrete compressions one-fifteenth as much, or 734 and 216 lb. per sq. in. Mr. Eddy.

A. W. BUEL, M. AM. SOC. C. E.—In the course of preparing a report on two buildings of the Turner "mushroom" type, then under construction, the speaker obtained from the Library of this Society, a search on this subject, brought down to October 17th, 1911. This search contained twenty-six references, of which about ten required careful consideration and comparison. Mr. Buel

The formulas and methods which had then been proposed as a basis for the design of flat slab floors gave results varying by as much as 300%, as illustrated by Table 3, which is taken from an article* by Mr. Louis F. Brayton.

TABLE 3.

No.	Method.	Thickness of slab, in inches.	Steel per panel, in pounds.
1...........	Cantilever........................	8	2 189
2...........	Turneaure and Maurer......	12	1 931
3...........	Grashof...........................	8	784
4...........	Mensch	8	2 120
5...........	Turner	8	549
6...........	McMillan	8	1 084
7...........	Brayton	8½	1 900

These methods, with the exception of Mr. Turner's, were not based on experiments of any consequence. In developing and checking his empirical method, Mr. Turner has made a considerable number of tests and experiments, but they are not entirely satisfactory or conclusive, because they are mostly confined to the loading of single bays, and because deformations or strains were not measured. They were sufficient, however, to show that something must have been neglected or erroneous data used in developing such formulas or methods as Nos. 1, 2, 4, and 7 of Table 3.

Mr. Arthur R. Lord has published† the results of his test of the slabs in the Deere and Webber Building, Minneapolis. Eight panels were loaded, and deformations were measured at twenty different points for the steel and about the same for the concrete.

Although neither Mr. Lord nor Arthur N. Talbot, M. Am. Soc. C. E., consider the data of this test sufficiently comprehensive to form the basis of a complete theory, they do give some valuable indications, and incidentally tend to confirm the empirical method of Mr. Turner.

**Engineering Record*, August 27th, 1910.

† First in *Engineering News*, December 22d, 1910, and recently reprinted in *Bulletin No. 64*, University of Illinois, Engineering Experiment Station.

Mr. Buel. The author's results seem to be in substantial agreement with Methods Nos. 1, 2, 4, and 7 of Table 3, but they do not agree with the tests. It is hardly conceivable that the author could have been familiar with Mr. Lord's tests when he wrote the paper. That the presentation of formulas or analyses of this kind should include a comparison with the best available experimental data hardly needs to be said, but the paper discloses no reference to any tests at all. The author's reasoning seems to be deductive, which, since the time of Lord Francis Bacon, has not been in favor for scientific investigations.

The amount of reduction in stresses due to arch and slab action in the Deere and Webber Building test was observed to be large, and, until this can be included in formulas, with at least approximate accuracy, it is little better than a waste of time to discuss slab theories. If Professor Eddy has succeeded in doing this, and from his past work we may be encouraged to look to him for results, slab design may be reduced to a rational basis, but, until this has been done, the best method to follow is that of empirical proportions from experiments, being careful to keep the variations from test slab dimensions within very narrow limits.

A little study of the comparative table of methods and a cursory comparison with the Deere and Webber test data would seem sufficiently convincing that the problem is by no means so simple as the author's statements imply. In fact, it is so complex that no one has yet been able to present a rational basis for design of flat slabs, unless Professor Eddy has succeeded in doing so in his recent work. Turneaure and Maurer's method (No. 2 of Table 3) was based almost entirely on Professor Eddy's previous work, with the value of Poisson's ratio assumed at 0.1. It now appears that Professor Eddy proposes to use a much larger value for this ratio as a result of more recent experiments. Turneaure and Maurer's method is at least 300% in error, according to the tests which have been made.

The speaker understood the author to say that theory is an explanation of facts, and that he liked to have his facts explained. The speaker has been unable to find a single fact in the paper, nor even an explanation of facts. It seems to be a paper of explanations, not only without facts, but contradicted by facts. The designers and builders of reinforced concrete structures have been waiting years for experimental facts on this slab question. We are beginning to get them, but we need at least twenty more such tests as that of the Deere and Webber Building before we will have a sound basis for a rational slab theory.

Should the discussion of this paper result in diverting some energy from theorizing toward the making of much needed scientific tests, the effort and space may not have been used in vain.

Mr. Walker.

E. G. WALKER, JUN. AM. SOC. C. E. (by letter).—The author's method of ascertaining a lower limit for the maximum stress in the steel of a reinforced concrete slab is of considerable interest. His assumption of absence of shear along the faces, *D, B, E,* and *C,* of the quarter slab (Fig. 1) seems to be quite justified in the case with which he deals; but, in practice, the use of a continuous slab supported on flare-topped columns is less general than the type of construction in which the floor panels are carried on secondary and main beams, which latter are supported by columns at intervals. In such a case, where the panel may be supported on four sides (Fig. 21), it may be assumed, using the author's reasoning, that there is no shear on the edges, *B* and *C,* of the quarter-panel. With reinforcement running parallel to both axes of the slab, and therefore supporting the slab along both edges, *D* and *A* (but principally along *D*), there must be shear on these two edges.

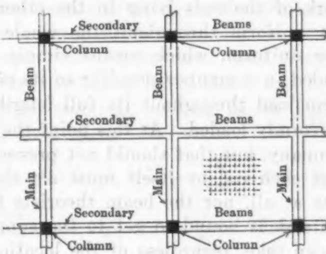


FIG. 21.

The writer would use the principles of Grashof's formula, or some equivalent method, in order to determine the relative values of these two shears, but would like to learn what the author's treatment would be in this case. The *raison d'être* of the paper is to ascertain a limiting value for the maximum stresses in the reinforcement, without the use of the more complicated and not wholly satisfactory assumptions of the slab formulas, but, for the moment, the writer does not see how, without their use, the author's method is to be applied to such cases as the one mentioned.

Mr Crehore.

WILLIAM W. CREHORE, M. AM. SOC. C. E.—In the opinion of the speaker, the observed differences between the author's beam theory and the actual results obtained in practice are due more to the cantilever action of the construction over the supporting columns than to tension in the concrete or to anything else.

The discussion of Mr. Nichols' very thoughtful paper seems to center about the method he uses in determining the statical limitation of the steel reinforcement in the floor slab—the point being raised by Mr. Turner that the so-called beam theory is not applicable to the calculation of a flat slab floor. It seems to the speaker that Mr. Nichols' answer to this objection is only partly effective, namely, that the direction of the stresses in the floor do not concern him, as he is dealing merely with the steel rod itself. It is quite true, and perfectly evi-

Mr. Crehore. dent, that the stresses affecting the rod in the concrete can act only in one direction, that is, in the line of its length; but it is equally true that, in the slab floor, these stresses are partly relieved by the work of the rods lying in the other direction, and that this relief is not uniform throughout the whole length of the rod, nor anything like uniform, which means simply that each individual rod will be loaded in a manner peculiar to its position, and will not receive a uniform load throughout its full length, although the floor slab itself be uniformly loaded. At this point the advocates of the two theories part company, but that should not prevent the slab theorists from realizing that each rod by itself must act the part of a beam, in so far as it acts at all, nor the beam theorists from recognizing that the load on the slab is bound to get to the supports by the shortest possible road it can take, regardless of the location of the rods.

Without attempting to decide between the respective merits of these two theories of computation, why should we not apply the beam theory as nearly in accordance with the assumptions of the case as it is possible to apply it, for the sake of giving this theory a fair test? The author seems to have stopped short of the complete application of the beam theory by not having taken into account the bending moments over the points of support. He assumes, and has just confirmed the assumption, that his hypothetical floor panel is fully loaded and that the load extends uniformly in all directions over the adjoining panels. Under such an assumption, a strict application of the beam theory requires a consideration of the moments developed over the points of support, that is, over the columns. This fact the author has not recognized. It is, however, vital to the assumptions made in this case; and, even if the adjacent floor panels were not loaded, it would still be necessary to take into account the moment over the supports due to their dead load, because of the continuous and homogeneous nature of the construction of the floor. This end moment or pier moment, in a girder with fixed ends (or the continuous girder, to which this case is analogous) is two-thirds as great as the moment developed at the center of an equal span having free ends, but, as it acts in the opposite direction, tending to hold up the central portion of the span, it should be deducted from the total moment due to a girder with free ends, such as the author has assumed when he writes $M = R_1 a$ as the bending moment due to all the external forces.

To recall a few elementary principles: Let the span, *L,* in Fig. 22, be one of the units of several equal spans, all uniformly loaded throughout with the unit load, *w,* per linear foot; and assume a beam of any material to rest continuously over the several supports so that the span in the figure is intermediate between loaded spans or panels on either side. Then the deformation of the beam in such a case would be concave downward for a distance from the support equal to about

Mr. Crehore.

one-quarter of the span (actually 0.21133 of the span) and concave upward in the center portion of the span. The effect of the reverse moments due to the fixed ends or cantilever action over the supports reaches out from the supports to these points of contrary flexure, and the actual resulting moment at the center is precisely the same as would be that of a span with free ends inserted between these points of contrary flexure. This is true for any series of equal spans and for any load within the elastic limit of the material, so long as the load is uniform throughout all the continuous spans.

The moment over the support due to the load, wL, is $\frac{wL^2}{12}$, and the moment at the center of the span is $\frac{wL^2}{8} - \frac{wL^2}{12} = \frac{wL^2}{24}$. Theoretically, then, under the conditions assumed, the greatest moment is to be found over the support, and is twice as great as the resulting mo-

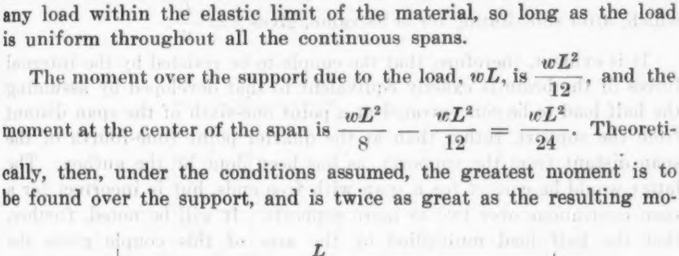


FIG. 22.

ment at the center; but, as points of maximum moment are also points of no shear, we do not find the greatest shear at the edge of the supporting column, where the author has assumed it to be, by his method of analysis. The greatest shearing force is to be found at a point where there is no moment, and in the present case such a point is in the plane, CF, of Fig. 22, where the curvature of the beam changes. At the point of contrary flexure in the plane, CF, the central portion of the spans hangs, therefore, just like a loaded span with free ends supported on the tip of a cantilever arm, which in reality the portion of the beam between CF and the support is.

Now, the length of the span between the two planes, CF, is known to be $\frac{L}{\sqrt{3}}$, and consequently the load hanging at the end of the cantilever arm in the plane, CF, is $\frac{wL}{2\sqrt{3}}$. For convenience, writing $m = 0.21133\,L$, which is the distance from CF to the nearest support,

Mr. Crehore.

we have, for the distance from the center of the support to the center of gravity of all the loads on the half span,

$$x = \frac{\frac{wmL}{2\sqrt{3}} + \frac{wm^2}{2}}{\frac{wL}{2\sqrt{3}} + wm} = \frac{mL + m^2\sqrt{3}}{L + 2m\sqrt{3}},$$

which, after substituting for m its value, gives $x = \frac{L}{6}$.

It is evident, therefore, that the couple to be resisted by the internal forces of the beam is exactly equivalent to that developed by assuming the half load to be concentrated at a point one-sixth of the span distant from the support, rather than at the quarter point (one-fourth of the span distant from the support), as has been done by the author. The latter would be correct for a span with free ends, but is incorrect for a span continuous over two or more supports. It will be noted, further, that the half load multiplied by the arm of this couple gives the moment, $\frac{wL^2}{12}$, which is known to be the maximum moment in the continuous-girder span uniformly loaded.

The application of this analysis to the case presented by the author is that the distance, *a,* of Fig. 2, between the forces, R_1 and R_2, forming the external load couple, should have been determined by treating the load of the quarter panel as if divided into two portions, which may be called the suspended and cantilever portions, respectively. If R_2 be located at the center of gravity of the quarter-column area, instead of at the center of gravity of the quadrant (as by the author's method), then the suspended portion of the panel's load should be assumed to be concentrated at a point situated 0.21133 of the diagonal distance between the centers of bearing. The author's distance, *a,* is not exactly the length of the quarter span between bearings, but is the definitely computed distance between the center of end shearing forces and the center of gravity of the quarter panel with the column area deducted. By the continuous-girder method, the arm, *a,* should be measured from the center of gravity of the quarter-column area to the center of gravity of the suspended portion of the load and the cantilever portion of the load combined.

If *S,* of Fig. 23, denotes the diagonal span between the centers of bearing, R_2 and R_2, and if m is 0.21133 of the diagonal span, *S,* then the suspended area of the panel is *DABCFE,* or the square, *OABC,* with the square, *ODEF,* deducted, and should be assumed to be concentrated at *E.* The cantilever area is the square, *ODEF,* with the quarter-column area deducted, and should be assumed to be concen-

trated at its center of gravity, G. To shorten the operation, it will be convenient to let Mr. Crehore.

$$y = m + \frac{4\,r\sqrt{2}}{3\,\pi}$$

the distance from the point, E, to the center of the column, in which expression r is the radius of the column. We then find the arm,

$$a = \frac{3\,L^2\,y - 3\,y^3 - 4\,r^3\sqrt{2}}{3\,L^2 - 3\,\pi\,r^2} - \frac{4\,r\sqrt{2}}{3\,\pi},$$

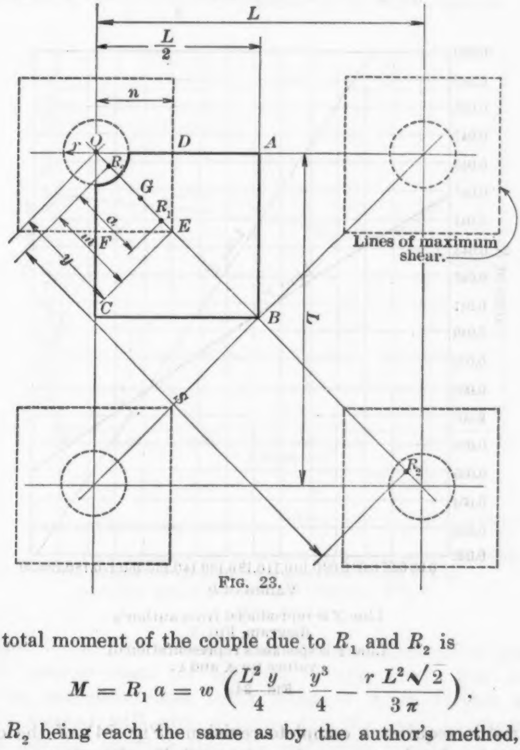


FIG. 23.

and the total moment of the couple due to R_1 and R_2 is

$$M = R_1\,a = w\left(\frac{L^2\,y}{4} - \frac{y^3}{4} - \frac{r\,L^2\sqrt{2}}{3\,\pi}\right),$$

R_1 and R_2 being each the same as by the author's method, namely, $w\left(\frac{L^2}{4} - \frac{\pi\,r^2}{4}\right)$. Substituting $y = n\sqrt{2}$, we have

$$M = w\left(\frac{L^2\,n}{4} - \frac{n^3}{2} - \frac{rL^2}{3\pi}\right)\sqrt{2},$$

Mr. Crehore.

from which, if $r = kL$, we obtain

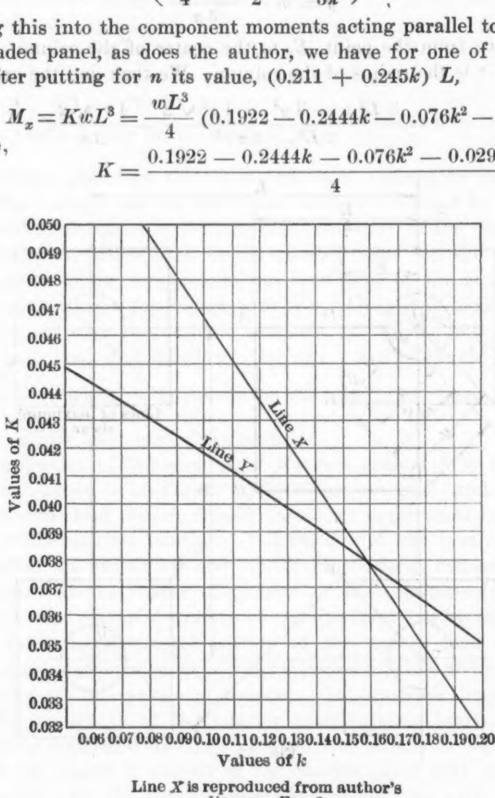
$$M = w\left(\frac{L^2\, n}{4} - \frac{n^3}{2} - \frac{kL^3}{3\pi}\right)\sqrt{2}\,,$$

Resolving this into the component moments acting parallel to the sides of the loaded panel, as does the author, we have for one of these moments, after putting for n its value, $(0.211 + 0.245k)\,L$,

$$M_x = KwL^3 = \frac{wL^3}{4}\,(0.1922 - 0.2444k - 0.076k^2 - 0.0294k^3).$$

Therefore,

$$K = \frac{0.1922 - 0.2444k - 0.076k^2 - 0.0294k^3}{4}.$$



Line X is reproduced from author's diagram, Fig. 3
Line Y is speaker's representation of values for K and k.

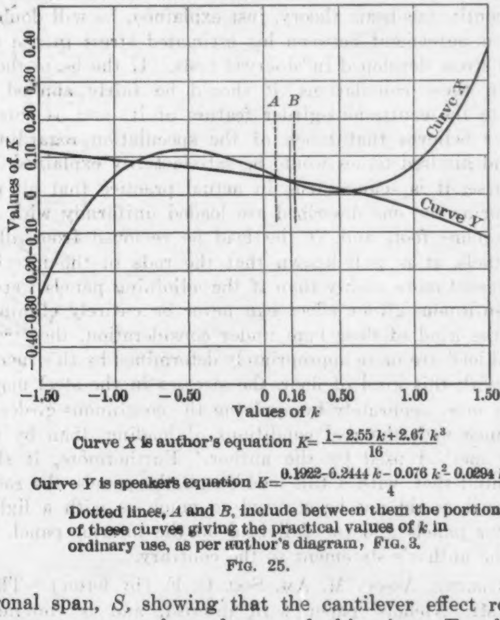
Fig. 24.

This result requires a complete revision (Fig. 24) of the diagram, Fig. 3, for the simultaneous value of k and K. The line on this diagram, although drawn as a straight line, is, of course, part of a curve (Fig. 25) known as the cubical parabola. So small a portion of the curve is needed between the limiting values used for k and K, that it is a straight line for all practical purposes. The speaker's equation for K

Mr. Crehore.

gives a line on this diagram crossing the author's line at about the value of $k = 0.16$, which means that, when the ratio of the column's radius to the length of the panel is 0.16, the arm, a, of the loading couple has the same length by each method. It should be observed that the author's distance, a, is measured from the center of gravity of the shearing forces along his section, A, of the panel, whereas the distance, a, by the continuous-girder method is measured from the center of bearing of the quarter panel.

Such an assumption, for the size of the column, as $k = 0.16$, gives the distance, $n = \frac{L}{4}$ (Fig. 23), although m still remains 0.21133 of



Curve X is author's equation $K = \frac{1 - 2.55\,k + 2.67\,k^3}{16}$

Curve Y is speakers equation $K = \frac{0.1922 - 0.2444\,k - 0.076\,k^2 - 0.0294\,k^3}{4}$

Dotted lines, A and B, include between them the portions of these curves giving the practical values of k in ordinary use, as per author's diagram, FIG. 3.

FIG. 25.

the diagonal span, S, showing that the cantilever effect reaches out the full quarter span for columns of this size. For larger columns, the suspended load being hung so much farther away, the speaker's method will give a value for the steel in excess of the author's method, showing that the cantilever method of construction is not economical for values of k greater than 0.16, and for all such the rods should be placed in the bottom of the slab only. Such a limit might have been inferred from a knowledge of cantilever bridges, as the length of the suspended span is determined there with regard to

Mr. Crehore. economy in the whole structure—if made too short, the cantilever spans will be excessively heavy, and if made too long the economy of that style of construction is sacrificed.

In the speaker's opinion, panels 6 ft. long, for example, between columns 2 ft. in diameter (or in that ratio) have their usefulness impaired by the size of the column. Values of k equal to or greater than 0.16 mean just this, and do not seem to be practical values in ordinary use.

Without referring to anything except the simple laws of mechanics of the beam, the speaker thinks he has justified "a result for the maximum stress in the steel smaller than the limiting value determined" in the paper. If the author will revise his calculations to conform with the continuous-beam theory, just explained, he will doubtless find much closer agreement between his estimated stress in the steel and the actual stress developed in observed tests. If the beam theory is to be used in these calculations, it should be fairly applied, without prejudice to the continuous-girder feature of it; and, if this is done, the speaker believes that much of the speculation regarding "dome action" and kindred topics would be satisfactorily explained.

Of course, it is seldom true in actual practice that all panels of such a floor as the one described are loaded uniformly with the same load per square foot, and, if the load be removed from all the adjoining panels, it is well known that the rods in the interior panel will be stressed more highly than if the adjoining panels were loaded; but the continuous-girder effect can never be entirely eliminated, because, in the kind of floor here under consideration, the stresses due to the dead load are more appropriately determined by this method. In any case, with this kind of floor, the stresses in the steel may be calculated far more accurately by applying the continuous-girder method, in accordance with the real conditions of loading, than by the free-end-girder method used by the author. Furthermore, it should be borne in mind that, with a full load on all the panels, the rods in the interior panel would not be stressed so much as with a lighter load on the outer panels and the full load on the interior panel, notwithstanding the author's statement to the contrary.

Mr. Greene. A. E. GREENE, Assoc. M. Am. Soc. C. E. (by letter).—The writer has read Mr. Nichols' paper with interest, and is convinced that the method used for determining the least possible amount of steel is correct. He does not fully grasp the criticisms of Messrs. Turner and Eddy, but gathers from their use of "true" and "apparent" stresses and the distinction of two kinds of bending moment, that they would use Ee instead of p as the stress in the rods. The writer maintains that such procedure would be quite incorrect.

There are a few textbooks which use the term, "true stress", for Ee, which is much better named the "equivalent simple stress", if

Mr. Greene.

a name for it is essential. The misleading nomenclature has been copied in the "American Civil Engineers' Pocket Book", and the statement is there made that in a steel bar under an axial pull of 6 000 lb. per sq. in., there is a "true" compressive stress of 2 000 lb. per sq. in., acting across the bar. If by this is meant that there is a resultant force of 2 000 lb. pressing against each square inch of a longitudinal plane through the bar, the statement is absurd. As the only force acting on the outer surface is that of the atmosphere, there is nothing to take up such stress at the surface. The idea probably arose from a misconception of Hooke's law of the proportionality of stress to strain. Hooke's law is true only between p_1 and e_1, when $p_2 = p_3 = 0$.

It is impossible to say what takes place in the bar, and, indeed, the engineer is interested only in the external phenomena, but, for the satisfaction of any one who may ask how lateral contraction can occur without transverse compression, the apparatus shown in Fig. 26 is presented to illustrate the possible interaction among the particles of an elastic solid. It consists of a number of balls connected by coiled springs in four directions. If the model is stretched vertically, it will contract laterally, but the resultant force on a vertical plane (that is, the stress on the plane) is zero and is of no avail in resisting external forces.

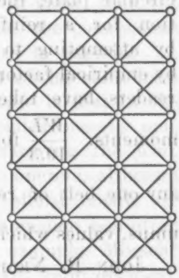


FIG. 26.

A symbol corresponding to f of Mr. Eddy's Equations (1) is not found in Grashof, nor does he anywhere, as far as the writer can determine, suggest that Ee is a stress in the body. If written in the form

$$f \leq Ee_1 = p_1 - K(p_2 - p_3),$$

in which f is the working stress, and p_1, p_2, and p_3 are the existing stresses in the material, it is simply a statement of the strain theory of failure of materials which has been followed generally by German writers. It is nothing but a theory, and merely means that the greatest strain in a material under compound stress should not exceed the strain caused in a bar by the working stress (which is conceived of as a simple stress), not that a stress of $p_1 - K(p_2 + p_3)$ actually exists in the material. It may be said, by the way, that recent investigations do not bear out the strain theory of failure, but seem to show that ductile materials, particularly soft steel, fail when the sheer reaches a critical amount, and that the criterion for designing in such materials should be $f \leq (p_1 - p_3)$, when $p_1 > p_2 > p_3$.

The fact that a floor slab is stressed in two directions does not mean that its resisting moment is to be computed from its strains, as may be seen from the following example: Imagine a rectangular bar of

Mr. Greene. some elastic material supported at its ends and loaded in between. Then $M = \frac{p_1 I}{y} = \frac{Ee_1 I}{y}$, in which M is the moment of the external forces. Now suppose a series of bars or plates are pressed against the sides by pairs of bolts, as shown in Fig. 27. The longitudinal strains are changed, but the stresses are not, for they must be sufficient to balance the external forces, which are the same as before, and the first part of the equation is still correct though the last part is not.

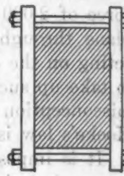


FIG. 27.

In view of the fact that a satisfactory mathematical solution has been obtained only for a homogeneous circular plate, the writer has no faith that any solution for a reinforced concrete slab will be found by attempting to analyze its strains. Moment equations corrected by empirical factors seem to him to offer the most logical solution. Some readers have taken Mr. Nichols' results on page 1678 as bending moments. $\frac{WL}{19.3}$ for Type V means a bending moment of at least $\frac{WL}{77}$ in any one belt of reinforcement at the center, and of $\frac{WL}{39}$ over the columns, values which certainly do not appear to be unreasonable.

Mr. Nichols. JOHN R. NICHOLS, ASSOC. M. AM. SOC. C. E. (by letter).—The results of this purely statical treatment of a statically indeterminate problem were, of course, not expected to establish actual values of the stress in the steel at any particular point. They do, however, establish clearly a certain limitation on the steel stresses.

The nature of this limitation is well illustrated in the case of a beam resting on three supports. We cannot determine the pressure on each support by statics alone, but statics does demonstrate that any computation, however plausible, which gives the total pressure on all three supports as less than the weight of the beam and its load is certainly wrong. Similarly, the writer considers that he has, by the use of statics, demonstrated that certain vigorously supported ideas as to the stresses in the steel reinforcement of a flat slab are certainly wrong.

Before replying to the questions raised in the several discussions the writer wishes once more to emphasize a point which seems to have escaped the attention of most of his critics, namely, that the results of the paper are dependent on only two arbitrary assumptions. One of these—that the shear is uniformly distributed about the periphery of the column cap—was stated specifically in the paper. The second assumption—in substance, that concrete is not to be relied on to afford tensile strength parallel to the main reinforcement—has practically universal acceptance in the analysis of rein-

forced concrete structures subject to bending. The adoption of this familiar assumption was implied though not stated in the paper. Mr. Nichols.

No one has questioned the substantial correctness of the assumption of uniform shear about the column cap, and no one has urged that reliance on "direct" tension in the concrete is permissible, although Mr. Eddy declares that what he calls "indirect" tension may be relied on. Neither has any error in the application of statics to the problem in hand been brought to light. What, then, is the basis of the objections to the paper? In every instance it boils down to the fact that the conclusions of the paper do not seem to agree with the results of tests. The critics do not attempt, however, to account for this, but content themselves with concluding, off-hand, that the paper is wrong; but the tests, after all, do not refute the conclusions of the paper. The apparent discrepancy is not hard to explain.

One reason why the stresses in the reinforcement of flat slabs have been found to be lower than the paper would indicate is that in the tests only a limited number of panels have been loaded, usually one, and adjoining unloaded panels have assisted in carrying the load. Another and far more important reason is that the concrete, although not avowedly relied on in tension, does actually afford tensile resistance, and materially assists the steel. This phenomenon has often been observed in tests of rectangular beams. For example, Mörsch* presents curves showing that in a beam having 0.4% reinforcement, a load which, according to computations based on no tension in the concrete, should have caused 16 000 lb. per sq. in. tension in the steel, actually produced less than 3 000 lb. per sq. in., as measured by extensometer. In a beam having 1.0% of steel, the measured tension was 8 500 lb. per sq. in. for a similar load. That such results should be expected can easily be shown by computation which takes into account the tension in the concrete. Table 4, computed on this basis and on the assumption that the straight line law of stress distribution holds for tension as well as for compression, gives the tensile stress in the steel, f_s, and the maximum tension in the concrete, f_t, for loads which, on the assumption of absence of tension in the concrete, would produce a steel stress of 16 000 lb. per sq. in. The depth to the steel is taken as eight-ninths of the total depth as, for instance, in a 9-in. slab,

$$\frac{E_s}{E_c} = 15.$$

Taylor and Thompson† give the modulus of rupture of plain 1:2:4 concrete as from 399 to 480 lb. per sq. in., the average of six tests being 439 lb. per sq. in.

* "Eisenbetonbau," 2d Edition (German), p. 101.

† "Concrete, Plain and Reinforced."

Mr. Nichols.

TABLE 4.

p	f_t	f_s
0.001	70	808
0.002	132	1 530
0.003	189	2 180
0.004	241	2 780
0.005	290	3 330
0.006	335	3 840
0.007	378	4 320
0.008	417	4 750
0.009	455	5 180
0.010	490	5 550

Tests by the United States Bureau of Standards* completely substantiate the evidence of the Mörsch tests and the table to the effect that concrete does, in fact, act in tension to an important extent to assist the main steel in simple beams. The extensometer tests, moreover, show that this phenomenon persists after cracks have appeared.

If tension in the concrete is thus capable of causing so pronounced a discrepancy between the results of beam tests and computations by the usual methods, the concrete of a flat slab must certainly reduce the measured stress in the embedded steel far below that which literal interpretation of ordinary analysis would lead one to expect. In flat slabs the concrete is stressed in multiple directions, so that here, if anywhere, "slab action" comes into play and Poisson's ratio has its use.

It seems perfectly clear that the two causes cited are quite sufficient to reconcile the conclusions of the paper with the results of tests. Furthermore, any alleged analysis of flat slabs, purporting not to rely on tension in the concrete, which, nevertheless, gives results agreeing closely with tests, and thus leaves no room for the actual tension in the concrete to show an apparent discrepancy, obviously, cannot be correct.

The question may now arise whether the tension in the concrete can properly be relied on. Mr. Eddy asserts that tension in the concrete of a flat slab is "indirect" and of the same nature as the diagonal tension which occurs in a beam. In the first place, the tension in question is not "indirect", but direct tension (if a distinction is to be made), occurring as it does on planes normal to the main reinforcement. In the second place, diagonal tension in beams is ordinarily not permitted to exceed 40 lb. per sq. in., unless steel is provided specifically to take the tension, and, even then, the diagonal tension is restricted in practice to 120 lb. per sq. in. There seems to be no good reason for permitting a higher value in flat slabs. If Mr. Eddy, therefore, wishes to utilize the tension in the concrete,

* Technologic Paper No. 2.

Mr. Nichols.

whether he styles it direct or indirect, he should, to be consistent, restrict his so-called "true stress" in the steel to something like $15 \times 120 = 1\,800$ lb. per sq. in. In that event, however, he might be informed that he is not "conversant with the commercial requirements" of flat slab design!

To put this question again, in slightly different form, ought tests, such as have been reported to date, to be adopted as a sole basis for flat slab design, regardless of the reliance on tensile strength of concrete which such design would involve? The writer believes they ought not. Considering that construction joints occur frequently in the middle of bays, shrinkage cracks and cracks due to loading, overloading and the possible effect of shock and fatigue, concrete tension cannot be depended on to be present and working when needed, any more than in beams or girders. The writer is prepared, however, to revise his opinion after a sufficient number of long-time fatigue tests and tests to destruction have yielded results which warrant him in so doing. Until such tests have been made, he cannot consider it good practice to place reliance on the tensile strength of the concrete, and any flat slab design which fails to meet the requirements developed in the paper is certainly dependent on such tension for its ability to carry load, whether or not the designer so intends. If we are ever to adopt the practice of counting on the tensile strength of concrete in beams or slabs, by all means let us do it with our eyes open, and without illusions.

Referring now to the several discussions, the writer is impressed with the apparent failure on the part of most, though by no means all, of those who have presented discussions to grasp the purpose and scope of the paper, and its method of procedure. It is hoped that the foregoing makes all this clearer and renders individual replies to the criticisms in the main unnecessary.

It may be worth while to consider Mr. Eddy's distinction between "apparent" stress and "true" stress. Mr. Eddy applies the term "apparent stress" to that which engineers commonly recognize by the name "stress," the stress measured by the force which would have to be applied at the section of a cut bar to hold one portion of the bar in equilibrium, the other portion having been removed. He dignifies by the name "true stress" a quantity of little more than academic importance, the stress within a material which would, in the absence of all transverse or lateral stress, produce the deformation existing in the material in the direction of the given stress. This misleading use of the terms "true" and "apparent" serves to cover up the assumption, which he makes without stating it, that failure of a material is dependent on its deformation. This theory, never more than a mere assumption, has been proven unfounded in tests made at Harvard

Mr. Nichols. University by Dr. P. W. Bridgman.* Yet this assumption is essential to the practical usefulness of Mr. Eddy's analysis of flat slabs, as presented in the discussion and, more fully, in his book "Flat Plate Theory of Reinforced Concrete Floor Slabs" to which Mr. Turner has referred. Without it, his work is without foundation.

Suppose, however, for argument's sake, that "true" stress, as defined by Mr. Eddy, were the criterion of failure of a steel rod. He gives the value of the true stress as

$$f_{s_1} = p_1 - Kp_2,$$

where p_1 is the "apparent" longitudinal unit stress in the rod, that is, the actual pull divided by the area of cross-section; and p_2 is the "apparent" lateral unit stress. Poisson's ratio, K, we will take at Mr. Eddy's liberal, if not wholly unwarrantable, value, 0.5. Let us now assign to p_2 another extremely liberal value, 500 lb. per sq. in., although it seems highly improbable that the concrete would, even if it could, exert so much lateral tension on the side of the rod. Then,

$$Kp_2 = 250.$$

From this it appears that the actual pull in the rod may reach a value of 16 250 lb. per sq. in. before the "true" stress exceeds the customary 16 000 lb. per sq. in. The difference is insignificant, even if it were valid.

Another highly vulnerable essential assumption in Mr. Eddy's analysis, however, is contained in the following quotation which appears both in the discussion and in his book:

"* * * and since we are evidently at liberty to assume our flat slab as sufficiently fine grained in its structure to act nearly like a slab constructed of some sort of homogeneous materials * * *"

This leads directly to his treatment of the reinforcing steel as equivalent to a uniform sheet of some metal which has a Poisson's ratio of 0.5.

A sheet of metal, however, is capable of resisting tension equally in all directions at the same time. To secure the same result with rods, obviously requires two layers at right angles aggregating twice the weight of metal contained in the sheet. This fact is entirely ignored in Mr. Eddy's analysis, and his consequent provision of steel is only half what his analysis, if logically applied, would call for, even if we could grant that it starts from valid premises.

Considering the errors that underlie the whole basis of Mr. Eddy's elaborate mathematical analysis of flat slabs, the results of which, though said to agree with certain tests, are widely at variance with the plain teachings of statics, we cannot take his analysis seriously. The agreement with tests, moreover, is largely fortuitous and, at

* *Philosophical Magazine*, July, 1912.

best, his results must be regarded as a sort of rule-of-thumb of a very limited and dubious range of application. Mr. Nichols.

Mr. Crehore's analysis seems to parallel the writer's presentation to some extent, but is based on assumed analogy with continuous beams. It has no advantage over the writer's analysis that he can discover, and, what is more, it seems to be numerically, at least, incorrect. This fact appears on comparing the writer's equation giving the value of K as

$$\frac{1 - 2.55\,k + 2.67\,k^3}{16}$$

with Mr. Crehore's

$$\frac{0.1922 - 0.2444\,k - 0.076\,k^2 - 0.0294\,k^3}{4}$$

In the extreme case when the supporting column caps become ideal points of support and $k = 0$, the writer's analysis would give the value of K as $\frac{1}{16}$, so that M_x, the sum of the moments on two opposite sides of the quarter panel, would be $\frac{W\,L^3}{16}$, as should be the case. Mr. Crehore's results would give K as

$$\frac{0.1922}{4} = \frac{1}{20.8}$$

which is obviously incorrect.

The writer wishes to take this occasion to offer an approximate method of applying the results of his paper, which is easily remembered and obviates the necessity of consulting the diagrams, as follows:

M_x is equal to the sum of the bending moments on two opposite sides of the quarter panel, including the component of the bending on the curved side. $2M_x$ is twice that sum, or the sum of the positive and negative bending moments, in the middle and at the side of a panel, respectively. It is easily shown that, approximately,

$$2\,M_x = \frac{w\,L\,l^2}{8}$$

where

$$l = L - \frac{2}{3}\,D,$$

D being the diameter of the column cap. The error involved in this approximation is less than 1% for values of $\frac{D}{L}$ less than 0.3.

In conclusion, it should be pointed out that this paper is not intended to be the final word in flat slab design. It is offered merely as

Mr. Nichols. a sound basis for a fair start. Beyond the realm of statics, and beyond the scope of this paper, lies the determination of actual stress distribution on the various cross-sections of the steel reinforcement; but the writer insists that, whether this determination is arrived at by the aid of reasonable assumptions, or by tests, or by computation based on elastic theory (and it must somehow be arrived at in practice), any method, the results of which fly in the face of statics, is certainly wrong.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1310

ADDRESS AT THE ANNUAL CONVENTION, IN BALTIMORE, MD., JUNE 2D, 1914.

BY HUNTER McDONALD, PRESIDENT, AM. SOC. C. E.

In accordance with the provisions of the Constitution, the President must deliver an Annual Address. It is not difficult to understand why such a provision was placed in the Constitution of earlier days. The publications of the Society were then the principal means for disseminating information on engineering subjects and exchanging views thereon. In the present day, however, each separate branch of the Engineering Profession is provided with its own special organ, published for profit, and they vie with each other in presenting accounts of engineering work in most attractive and readable form. Society journals have become mere repositories of facts and opinions, and are not often read voluntarily, but are usually carefully preserved and are invaluable as sources of reference by reason of their authentic character and prestige. We should carefully guard and preserve the high quality of our publications, and all papers intended to promote private interests commercially should be denied admission.

In the earlier days the President was charged with the duty of reviewing the engineering progress of the previous year. A literal compliance with such a provision now would be a stupendous task, and, even in the hands of a skilled and entertaining author, would be uninteresting to the members because they have seen it all in such very attractive form from week to week in engineering journals.

Fortunately for me, and doubly fortunate for my audience, the character of the presidential address is no longer prescribed, and any amount of latitude seems permissible; at least such seems to be a reasonable inference from a cursory examination of some of the addresses of my predecessors in office.

Taking advantage of such liberty, and having due regard for the patience of my audience, I shall briefly refer to some of the more important matters of interest which the present year brings to engineers, and shall then direct attention to some matters concerning the welfare of our Society and the profession of which it is the foremost exponent.

It is a matter of pride and cause for congratulation that the most stupendous work of civilization, the Panama Canal, is about to be presented to our Government by men of our profession, completed greatly in advance of the time when it was due, and absolutely free from any taint of scandal, graft, or incompetency in any of the stages of its progress.

Fresh from the glories of this achievement, our Government has entered upon two more gigantic tasks to be performed in untried fields. These are the valuation of the railroad and telegraph properties of the United States, and the building of railroads in Alaska. The first of these projects is engaging the thought and time of a large number of our profession, and has already resulted in greatly increasing the demand for and compensation of engineers at a time when, on account of general stagnation in industrial development, many would otherwise be idle. In this work engineers have the opportunity to show to the country that they, more than any politicians or lawyers, are qualified to ascertain and fix the underlying principles upon which this valuation should be made, and then to follow this up by the actual working out of the results. It is a matter of encouragement to our brethren—although probably not to those of our citizens at large, who have given the matter much thought—to reflect, that from five to ten years will be required to complete the task, and, when it is done, a large force of engineers must be constantly employed, both by the railways and the Government, to keep the work up to date. It is not for us to say whether or not "the game is worth the candle", but to address ourselves to the task set before us and demonstrate our fitness for the responsibility.

The science of such valuation is new. In 1897 it became my duty to serve as one of a committee to value a piece of railroad property for a joint terminal. Being inexperienced, I sought from the pages and editors of technical journals and from other engineers some information as to the basis for such valuation. I was unable at that time to secure it. Now there is so much in print on the subject that is with difficulty that the chaff can be separated from the grain. We have a Special Committee which is wrestling with this problem, and its preliminary reports indicate that important results are to be expected from its labors.

A commission of engineers is now charged with the duty of selecting a route for the main line of a railroad from some harbor on the southern coast to the central part of Alaska, and to determine what and where branches are to be built. When the location is determined, it is probable that the building of the roads will be entrusted to the United States Corps of Engineers. It seems improbable that the roads will ever be operated by any agency other than the National Government. An opportunity presents itself here for civil engineers to demonstrate their fitness for responsibilities other than the mere application of mathematics. A civil engineer is now Governor of the Canal Zone, and another should be General Manager of the Alaskan Railroad when the time for its operation arrives. It is not necessary to state that his staff should be composed of engineers. An engineer has recently been chosen as manager of one of our large cities. Many are being placed on City Commissions.

Persons who are charged with the responsibility for the wise expenditure of public money are more and more inclined to turn to the engineer for advice as to how to do it, and to recognize the fact that civil engineers are fit for something else than preparing blue prints, setting stakes, and running instruments. We are now on the threshold of new responsibilities, and we must set ourselves faithfully to the task of preparing to meet them. Good engineering consists largely in meeting emergencies with the resources at one's disposal. The man who does it is always in demand.

Much of the space in the addresses of some of our Past-Presidents has been devoted to discussing the status of the engineer, the ethics of the profession, and how to compel an apparently unwilling public to recognize the fact that engineering is a profession. Complaint is

rife that we do not get our just deserts, and that something not clearly defined must be done to remedy this injustice.

I have been tempted to delve somewhat deeper into the origin and present meaning of the word "engineer" than my predecessors in office have done. Realizing my own lack of fitness for the task, as far as the origin is concerned, I called in a specialist on etymology, Dr. Bert E. Young, Professor of Romance Languages of Vanderbilt University, and lately honored by the French Government with the decoration of Officer of Public Instruction for distinguished services in the promotion of French literature.

I quote his statement in full:

"I take pleasure in furnishing the best information I have as to the origin of the word 'engineer':

"*Engineer,* formerly *enginer,* or sometimes *ingener,* Middle English *engyneour,* from the Old French *engignier* or *engigneour,* or shorter *engineur.* This from Middle Latin *ingeniarius,* one who makes or uses an engine, especially a war engine. *Engine* is from Latin *ingenium,* an invention, an engine.

"The French word *ingenieur,* a semi-learned form, seems to come down by a slightly different road, being from the hypothetical Middle Latin word *ingeniator* from *ingeniare* from *ingenium,* an engine. Or else the Modern French *ingenieur* is an alteration of the better form *engigneur,* mentioned in the first paragraph.

"*Genie* from Lat. genius, orig. a spirit, extended as a term of war to the sense of *art* of attacking or defending fortified places, etc., and then to the arm of the service devoted to military or naval engineering.

"*Ingenuity* from French *ingenuite* from Lat. *ingenuita (t)s* from *ingenuus,* native, born of free parents, frank, generous. *Ingenious* from Lat. *ingeniosus,* gifted with genius or natural capacity. Lat. *ingenium* from Old Latin *genere.*

"Apparently all go back to the ubiquitous root, $\sqrt{\text{gen}}$, to bear, to produce, to give birth to.

"Misc. remarks:

"Considering the derivation of ingenuity, there must have been confusion of Lat. *ingenuus* and Lat. *ingeniosus.* These should be almost opposite in meaning. I suppose an *engineer* ought to be both *ingenious* and *ingenuous,* artful and artless, sophisticated and unsophisticated, bond and free. According to the above, I should say that the man who pronounces *injine* is only atavistic.

"Ex.—'In hys court was a false traytoure, that was a grete Yngynore (plotter). (Halliwell, 1420).'

"Ex.—'The most prime Engineers of Oaths that ever the world knew. (Butler, 1680)'.

"Ex.—'For 'tis the sport to have the engineer hoist with his own petar. (Shak., 1602, Ham. iii, 4, 206).'

"Ex.—'The Ingener of this practice—was a Portugal. (Carew, 1620)."

From the Oxford Dictionary, which no doubt gives the prevailing English view, I have extracted the following definitions and examples of the use of the word "Engineer".

"One who contrives, designs or invents. Ex. 1611, Rich, Honest age, 1844 (36) 'Yet you cannot deny them to be the devil's engineers'.

"1635, R. Gibbs in Spurgeon 'That great engineer Satan'.

"A constructor of military engines (Obsolete). One who designs and constructs military works for attack or defense. 1325 Coer De Leon 1387—'a tour ful strong, that queyntyly engynours made'.

"A soldier belonging to the division of the army called engineers, composed of men trained in engineering work. Ex., Porter's History, Royal Engineers. 'This day (May 26th, 1716) may thereafter be taken as the one on which the engineer branch blossomed into a distinct corps.'

"One whose profession is the designing and constructing of works of public utility, such as bridges, roads, canals, railways, harbors, drainage works, gas and water works. From the 18th Century, also civil engineers (from the military).

"Ex. 1792 Smeaton Reports (1797) I pref. 7. 'The first meeting of this new constitution, the Society of Civil Engineers, was held on the 15th of April 1793.' Ex. Eddyston L. introduction—'My profession of a civil engineer.'

"A contriver or maker of engines (specifically mechanical engineer).

"One who has charge of a steam engine. In England only with reference to marine engines. In the United States often applied to the driver of a locomotive. Ex. 1860, Bartlett's Dictionary. American, Engineering, 'the engine driver on our railroads is thus grandiloquently designated'.

"Engineman, one who works or helps to work a fire engine. One who attends to a stationary engine. The driver of a locomotive."

From other English authorities:

March's Thesaurus Dictionary. "One versed in any branch of engineering. To manage skillfully. The science and art of building, making and using engines and machines."

Hallowell's archaic and provincial words. "Engin, wit, contrivance (Lat.). Inginous, inventive (Jonson)."

From Joseph Wright's English Dialect Dictionary. "In 1649 Gray wrote 'Master Beaumont a gentleman of great ingenuity and rare parts adventured into our mines, who brought with him many rare engins. The memory of these 'rare engins' survives in the name of the seam

which he appears to have discovered, still called the engin seam." It will be noted that Beaumont was practicing civil engineering at quite an early date.

From the Century Dictionary, presenting the American view, the following:

"A person skilled in the principles and practice of any department of engineering. Engineers are classified according to the particular business pursued by them, as *military*, *naval* or *marine*, *civil*, *mining*, *mechanical* and *electrical* engineers.

"To work upon; ply; try some scheme or plan upon.

"Ex.—'Unless we *engineered* him with question after question, we could get nothing out of him.'—*Cowper*.

"To guide or manage by ingenuity or tact; conduct through or over obstacles by contrivance or effort; as to *engineer* a bill through Congress."

Through the kindness of our Secretary, Mr. Hunt, I have been able to secure the following translations from the Dictionnaire de la Langue Française:

"One who plans, lays out and directs field work and fortifications, for attacking, defending or fortifying places.

"Ex.—'Then he (Peter the Great) has had even a school of engineers in a country where no one (before him) knew the elements of geometry, Volt. *Charles XII*, I.'

"One who directs [private] works or public works, such as the construction and maintenance of roads and bridges, working of mines, etc. Engineer of bridges and roads. Mining Engineer, Naval Engineer or Marine Engineer, Railroad Engineer.

"Civil Engineer, name given to engineers who are not trained in technical schools or who work for private industry.

"Naval Architect or Engineer, one who applies himself to the art of naval construction.

"Topographical Engineer, one who draws maps. Hydrographical Engineer, See Hydrography.

"Engineer for mathematical instruments, one who makes mathematical instruments."

Also, from Cotgrave's Dictionary of the French and English tongue:

"Enginier —An engineer; a maker of engins.
Enginer —Engigner.
Engigner —A deceiver, beguiler, cousener; also an enchanter.
Engin —An engin, toole, instrument; also understanding, policie, reach of wit; also suttletie, fraud, craft, wiliness, deceit.
Ingenieur—An enginer, engine-maker; fortifier."

Mr. Hunt also furnished me the following translation from Grimm's Deutsches Wörterbuch:

"Engineer, n.—This word of the present day, for military architect, surveyor, introduced from a foreign language in the 17th century, represented an artful calculating person in general. Ex.—'He who wishes to bring Germans into an understanding must be a very good and wise engineer (schemer, calculator)'. Weidner's, Zinkgres, 3, 158."

No trace of the use of the word "Engineer" or of any of the words from which it is derived in describing the building of any of the ancient structures has been found. That engineers of great skill existed is evidenced by monumental and written records, but they were designated otherwise, as for example: Frontinus, in his water supply of the City of Rome, as translated by Clemens Herschel, M. Am. Soc. C. E., mentions *moduli* (levelers), *scientia* (builders), *agrimensores* and *geometricii* (land surveyors), *architectos* (architects), and *Curator Aquarum* (Commissioner of Water Works), but none of the Latin words from which the word "engineer" is derived, are used in connection with the process of building or maintaining. The conclusion is inevitable that the name at least is comparatively modern, and that the attempt to class the calling among the learned professions is still more so.

John Smeaton was the first engineer to style himself Civil Engineer (probably in 1782).*

In the Annual Address of Mr. Charles Hawksley, President of the Institution of Civil Engineers, 1901, an extensive quotation is made from the Report of the Council, for the Session, 1885-86. A portion only is repeated here:

"'The honor of having originated the Institution is often assigned to Smeaton, or to Telford, but the idea is erroneous in both cases. Smeaton died many years before it was thought of, and Telford only joined it after its establishment.'

"A Society of Engineers, still existing, was founded by Smeaton in 1771; it includes many of the most eminent members of the profession, but it is rather of the nature of a social club than of a scientific association, and has no connection with this Institution."

To those who are interested in the beginnings of engineering, I would recommend an examination of the two papers above referred

*See discussion by Clemens Herschel, M. Am. Soc. C. E., of Paper No. 472, entitled "Beginnings of Engineering", by the late J. E. Watkins, Assoc. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XXIV, p. 382.

to and the following: Annual addresses of Past-Presidents William E. Worthen, 1887 (*Transactions,* Vol. XVII), and William P. Shinn, 1890 (*Transactions,* Vol. XXII), also of Charles B. Vignoles, F. R. S., Past-President, Institution of Civil Engineers, 1870 (*Minutes of Proceedings,* Vol. XXIX).

On page 604 of *Engineering News,* May 26th, 1910, is an entertaining paper entitled the "Origin and History of the Profession of Civil Engineering". A foot-note describing it reads as follows:

"An appendix to the Presidential address of Jas. C. Inglis, President of the Institution of Civil Engineers of Great Britain. Abridged from the account contained in the life of Sir William Fairbairn by the late Dr. William Pole, F. R. S., Hon. Secretary of the Institution."

A portion of the beginning is quoted:

"The root of both the words, Engineer and Engine, is found in the Sanscrit *jan,* to be born, from which came the Greek form *γεν,* and the Latin *gen,* the latter being embodied in the old verb *genere,* with its compound *ingenere* (changed into *ingignere*), to implant by birth, and in their later substantive *ingenium,* an innate or natural quality.

"The old Latin verbs, *genere* and *ingenere,* gave rise to the French form, also a verb, *s'ingenier.* This is of great antiquity, and from its comprehensive and useful meaning it has continued in use down to the present time, being found continually in modern French writings. The import of it has nothing whatever to do with engines or machines, but is purely psychological. It is given in Littre's great French dictionary:

"'*Chercher dans son genie, dans son esprit, quelque moyen pour reussir.*'

"Now, all authorities, including our own great Lexicographer, agree that this word is the true origin of the word 'Engineer', and thus we arrive at the interesting and certainly little-known fact that an Engineer is, according to the strict derivation of the term, not necessarily a person who has to do with engines, but any one who seeks in his mind; who sets his mental power in action, in order to discover or devise some means of succeeding in a difficult task he may have to perform."

There are sufficient divergences of authority and definitions in the above quotations to give room for any amount of argument. I will be content with simply recording them.

These matters are mentioned primarily as leading up to a consideration of the condition in which the profession in this country

now finds itself. There is ample evidence that the word "Engineer" is now applied alike to the designers of engineering works and to the operators of engines. It is also evident that civil engineering is now considered by many to be a branch of engineering. In the language of the late Grover Cleveland "It is a condition and not a theory which confronts us." We could not, if we desired, compel the public to call locomotive engineers "enginemen" or "engine drivers", nor stationary engineers, "power superintendents", notwithstanding the fact that such they are. President Worthen in the address above referred to says:

"I would drop the term civil as embracing too large a profession, and adopt prefixes, as military, hydraulic, railway, mechanical, steam, electric and other divisions, when these are adopted as specialties."

His idea seems to have been merely to divide civil engineers into different classes. He does not mention civil engineers as one of the classes.

Until recently, I have also been inclined to the view that we should drop the word "Civil" from the name of our Society, but a closer study of the matter confirms me in the opinion that we should retain it. I think there can be no question but that civil engineering embraces all engineering except that connected with the actual conduct of war. The fact that the military engineer "in piping times of peace" performs the work of the civil engineer constitutes no distinction.

Conscious of our pre-eminence, this Society has been content to sit still and let the branches of our profession grow up independently until they have become of such importance as to force us to accept the meaning of the word "Civil Engineer" as indicating a specialty or division of the profession. This is a false position, and we alone are responsible for it. Smeaton evidently intended to make it a specialty, but he could not have known, at the time he referred to it, how intricate the divisions of the specialty were to become. Had we pursued a course similar to that of the engineers of Germany we would not to-day find ourselves in the humiliating position of being asked to substitute the word "Professional" for the word "Civil" in the bill recently pending before the New York Legislature for the regulation of the practice of civil engineering, before the other societies would co-operate with us in securing a bill which would be acceptable to the entire profession.

Believing that it would be interesting to know the number of the various kinds of engineers which go to make up our membership, I requested the Secretary to prepare from the List of Members a classification. After much labor it has been accomplished and is presented herewith in tabular form. It will be noted that in this classification our Secretary treats civil engineers as one of the divisions or classes. Under the conditions, he could not do otherwise.

I have alluded to the status of the profession in Germany, and will now describe its condition more at length, also referring briefly to conditions that exist in a few other countries. This is done with a view of comparing conditions in these countries with those in our own.

My information regarding Germany is taken from an article in *Engineering,* London, December 20th, 1912, entitled "The Verein Deutscher Ingenieure". It was founded in 1856. Quoting verbatim, the dominating ideas are expressed under three heads:

"1. The society must in its activities embrace the whole of Germany. [This is remarkable in that it anticipated Bismarck's federation by 15 years.]

"2. Local sections (*Bezirksverein*) must be founded in all parts of Germany.

"3. A good journal must be published."

In 1865 it had 1 000 members; in 1875, 3 000; in 1885, 5 300; in 1895, more than 10 000; and in 1912, about 24 000.

It has for its object, as indicated by the printed statutes, the intimate co-operation of the intellectual forces of German technical science for the benefit of the entire industries of the Fatherland. This object is to be attained by meetings of the entire Verein and its local sections, the *Journal,* the establishment of libraries and reading rooms, the editing of important technical works, the instituting of engineering research on subjects of industrial importance. The membership embraces (*a*) engineers, and (*b*) those capable of advancing the objects of the Union. Contributions are made from the general fund toward the support of local sections. All members living within the Empire must belong to local sections. In one year 300 business meetings were held and 400 lectures on technical subjects were delivered. The question box is in use at all sections, and a general technical committee answers all questions not answered by the members of the section at the regular meeting. These local sections, as a rule, are in touch

with the educational and industrial powers of the neighborhood. Courses of instruction and excursions through works are arranged for, and visits are frequently exchanged between sections. "Sentiment plays a much more prominent part in the life of the local sections than English [and, I may say, American] prejudice would allow." Honors are bestowed on leading members as they reach advanced age. and "the social graces are not neglected." Headquarters, libraries, and reading rooms are often in hotels, to the mutual benefit of the section and the proprietor. Weekly meetings are held with tables reserved. Ladies attend the monthly meetings. Play on the skittle alley, and free and easy beer evenings are monthly features of some sections. The Board of Management of seven is selected by delegates from the sections, and this council controls, except on extraordinary occasions when they meet with the delegates. Local sections are self-governing, provided their constitutions are approved by the council and their actions further the society's objects. General annual meetings are held. The backbone of the organization is the *Journal.* In 1911 it had 2 200 pages of text, exclusive of the advertisements. News and proceedings of local sections are published. Books are reviewed with conscientiousness. Brief abstracts are given of important articles in more than 60 leading journals. They follow closely on the article itself, and are then reprinted on one side of the sheet, with room for marginal notes. Much space is devoted to correspondence. Original researches are encouraged, and grants are made from the general fund therefor. Medals are awarded for specially good work, and bad work is not permitted. In the supplement to the *Journal,* questions of financial, political, economical, and literary news are treated. The general aim of the supplement is to afford the engineer knowledge of the wider world which he serves, and to bring the needs of the engineering world before the sociologists, economists, and politicians. "The enormous number of members and the influential positions which many of them hold, make the Verein a power in the land, with a voice for the industrial elements such as no other country possesses." It has had great influence on technical education, and has banished Latin and Greek from the curriculum.

A new movement to inject engineering ideas into municipal and State administrations has been inaugurated and is meeting with great success. The framing of patent laws and the purification of

Associate Members.																			
Consulting Engineers	1	....	2	1	7	....	229	3	2	9	38	17	12	2	3	....	14	340	13.9
Civil Engineers	....	4	....	....	8	....	....	....	....	....	....	....	....	....	....	....	....	12	0.5
Chief Engineers	23	9	44	73	110	....	....	....	....	....	....	....	....	....	....	....	....	259	10.6
Assistant Engineers	246	157	162	232	405	1	....	....	....	....	....	....	....	....	....	....	....	1 208	49.2
Instrumentmen, etc.	6	31	....	7	11	....	....	....	....	....	....	....	....	....	....	....	....	55	2.3
Presidents, Managers, Superintendents, and Other Officials	19	26	8	14	255	3	....	....	....	....	....	....	....	....	....	....	....	325	13.3
Professors and Associate Professors	....	....	....	....	....	58	....	....	....	....	....	....	....	....	....	....	....	58	2.4
Assistant Professors and Instructors	....	....	....	....	....	43	....	....	....	....	....	....	....	....	....	....	....	43	1.8
Contracting Engineers	....	....	....	....	16	....	80	....	....	....	....	3	....	....	....	....	3	102	4.1
Corps of Engineers	....	10	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	10	0.4
Other Military Engineers	....	2	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	2	0.1
Architects	....	....	....	1	....	....	14	....	....	....	....	....	....	....	....	....	....	15	0.6
Commercial Representatives	....	....	....	....	13	....	....	....	....	....	....	....	....	....	....	....	....	13	0.5
Miscellaneous	....	1	....	....	6	....	....	....	....	....	....	....	....	....	....	....	....	7	0.3
Total	295	240	216	328	831	105	323	3	2	9	38	20	12	2	3	....	17	2 444	....
Percentage	12.1	9.8	8.8	13.4	34.0	4.3	13.2	0.1	0.1	0.4	1.6	0.8	0.5	0.1	0.1	....	0.7	....	100
Associates.																			
Consulting Engineers	....	....	....	....	2	....	3	....	....	2	....	1	....	5	....	....	1	14	12.4
Assistant Engineers	2	4	....	5	10	....	....	....	....	....	....	....	....	....	....	....	....	21	18.6
Instrumentmen, etc.	....	1	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	1	0.9
Presidents, Managers, Superintendents, and Other Officials	4	1	....	....	34	....	....	....	....	....	....	....	....	....	....	....	....	39	34.5
Professors and Associate Professors	....	....	....	....	....	10	....	....	....	....	....	....	....	....	....	....	....	10	8.8
Assistant Professors and Instructors	....	....	....	....	....	4	....	....	....	....	....	....	....	....	....	....	....	4	3.5
Contracting Engineers	....	....	....	....	....	....	8	....	....	....	....	1	....	....	....	....	....	9	8.0
Architects	....	....	....	1	....	....	1	....	....	....	....	....	....	....	....	....	....	2	1.8
Commercial Representatives	....	....	....	....	7	....	....	....	....	....	....	....	....	....	....	....	....	7	6.2
Miscellaneous	1	1	....	....	3	....	....	....	....	....	....	....	....	....	....	....	1	6	5.3
Total	7	7	....	6	56	14	12	....	....	2	....	2	....	5	....	....	2	113	....
Percentage	6.2	6.2	....	5.3	49.5	12.4	10.6	....	....	1.8	....	1.8	....	4.4	....	....	1.8	....	100

Character of Employment of the Membership of The American Society of Civil Engineers.

In the List of Members, February 10th, 1914, a large percentage of the total membership (19.72) do not give any specific occupation, and therefore cannot be classed. The numbers and percentages in the following Table, therefore, are based on the 5 864 whose occupation is given in such a way as to enable them to be classed at least with some degree of probable accuracy.

		Character of Employment.						Private Practice.												
		Railroads.	National Governments.	States and Counties.	Municipalities.	Private Companies and Individuals.	Technical Schools.	General.	Mechanical.	Electrical.	Sanitary.	Hydraulic.	Structural.	Mining.	Chemical and Metallurgical.	Highway.	Heating and Ventilating.	Miscellaneous.	Total.	Percentage.
Hon. Members.	Consulting Engineers	1																	1	33.3
	Civil Engineers					1													1	33.3
	Corps of Engineers		1																1	33.3
	Total	1	1			1													3	
	Percentage	33.3	33.3			33.3														100
Cor. Members.	Consulting Engineers							1											1	
	Total							1											1	
	Percentage							100												100
Members.	Consulting Engineers	5	3	4	11	18		545	10	1	15	59	17	19	8	2	2	19	738	27.0
	Civil Engineers	2	26			7													35	1.3
	Chief Engineers	102	14	54	124	125													419	15.6
	Assistant Engineers	200	163	61	163	183													770	28.5
	Instrumentmen, etc.	1	4																5	0.2
	Presidents, Managers, Superintendents, and Other Officials	72	12	5	19	284													392	14.5
	Professors and Associate Professors						110												110	4.1
	Assistant Professors and Instructors						6												6	0.2
	Contracting Engineers					11		92					11					2	116	4.3
	Corps of Engineers		55																55	2.0
	Architects	1	1					36										1	39	1.4
	Commercial Representatives					15													15	0.6
	Miscellaneous	1	2		1	4													8	0.3
	Total	384	280	124	318	647	116	673	10	1	15	59	28	19	3	2	2	22	2 703	......
	Percentage	14.2	10.3	4.6	11.8	23.9	4.3	24.9	0.4	0	0.6	2.2	1.0	0.7	0.1	0.1	0.1	0.8	......	100

RECAPITULATION.

TOTAL MEMBERSHIP.	CHARACTER OF EMPLOYMENT.						PRIVATE PRACTICE.											Total.	Percentage.
	Railroads.	National Governments.	States and Counties.	Municipalities.	Private Companies and Individuals.	Technical Schools.	General.	Mechanical.	Electrical.	Sanitary.	Hydraulic.	Structural.	Mining.	Chemical and Metallurgical.	Highway.	Heating and Ventilating.	Miscellaneous.		
Consulting Engineers	7	3	6	12	28		790	13	3	28	99	36	32	10	5	2	35	1 109	18.9
Civil Engineers	2	33	1		16													52	0.9
Chief Engineers	125	23	106	210	252													716	12.2
Assistant Engineers	496	344	271	456	763	2												2 332	39.8
Instrumentmen, etc.	21	65	14	18	39	1												158	2.7
Presidents, Managers, Superintendents, and Other Officials	97	39	13	33	613	3												798	13.6
Professors and Associate Professors						188												188	3.2
Assistant Professors and Instructors						77												77	1.3
Contracting Engineers					30		191					15					5	241	4.1
Corps of Engineers		70																70	1.2
Other Military Engineers		5																5	0.1
Architects	1	1		2			51										1	56	0.9
Commercial Representatives					38													38	[illegible]
Miscellaneous	3	4		1	14												2	24	0.4
Total	752	587	411	732	1 793	271	1 032	13	3	28	99	51	32	10	5	2	43	5 864	......
Percentage	12.8	10.0	7.0	12.5	30.6	4.6	17.6	0.2	0.1	0.5	1.7	0.9	0.5	0.2	0.1	0	0.7	......	100

CHARACTER OF EMPLOYMENT OF THE MEMBERSHIP OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.—(*Continued.*)

	Character of Employment.						Private Practice.												
	Railroads.	National Governments.	States and Counties.	Municipalities.	Private Companies and Individuals.	Technical Schools.	General.	Mechanical.	Electrical.	Sanitary.	Hydraulic.	Structural.	Mining.	Chemical and Metallurgical.	Highway.	Heating and Ventilating.	Miscellaneous.	Total.	Percentage.
Juniors.																			
Consulting Engineers					1		12			2	2	1	1				1	20	3.4
Civil Engineers		3	1															4	0.7
Chief Engineers			8	13	17													38	6.4
Assistant Engineers	48	20	48	56	165	1												338	57.5
Instrumentmen, etc.	14	29	14	11	28	1												97	16.5
Presidents, Managers, Superintendents, and Other Officials	1				30													31	5.2
Professors and Associate Professors						10												10	1.7
Assistant Professors and Instructors						24												24	4.1
Contracting Engineers						3	11											14	2.4
Corps of Engineers		4																4	0.7
Other Military Engineers		3																3	0.5
Commercial Representatives					3													3	0.5
Miscellaneous	1				1													2	0.4
Total	64	59	71	80	245	39	23			2	2	1	1				1	588	
Percentage	10.9	10.0	12.1	13.6	41.7	6.6	3.9			0.3	0.3	0.2	0.2				0.2		100
Fellows.																			
Presidents, Managers, Superintendents, and Other Officials	1				10													11	91.7
Miscellaneous																	1	1	8.3
Total	1				10												1	12	
Percentage	8.3				83.4												8.3		100

technical language has received attention. It has fixed mechanical standards as well as fees for engineers and architects. Needy members are assisted with charity. No correspondence tuition is allowed to be advertised in the journal. Branches in foreign countries are contemplated.

I have dwelt at considerable length on the abstract of this article because it shows what might have been accomplished in this country had the management of our Society shown the same interest in the profession at large as it has in the welfare of its members only.

In France the good of the entire profession has always been the object of the Société des Ingenieurs Civils, which is the title of the organization of the free engineers as distinguished from that of the engineers of the Ponts et Chaussées and other Government engineers. This Society was founded in 1848. It flourished from the beginning, and received many donations. In 1877 its new building was completed, and the Society moved into it. Government engineers are not eligible, but it was organized largely by Alumni of the Government schools. Its avowed objects are:

1. To throw light by discussion on obscure questions in civil engineering;
2. To assist in aiding the development of applied science auxiliary to civil engineering and to assist industries;
3. To extend professional teaching among workmen and shop foremen;
4. To investigate questions of industrial economy of administration, to increase the power production and wealth of the country;
5. To insure close relations among its members;
6. To act as a kind of employment bureau for its members; and
7. To establish a fund in case of necessity.

An examination of the year-book of the Society indicates a very prosperous condition, both as to finances and membership. The members are scattered over many colonies and foreign countries. Sections exist in all provinces.

The following is quoted from the List of Members of the Institution of Civil Engineers of Great Britain:

"The Institution of Civil Engineers was first established, and has since been incorporated by Royal Charter, for the General Advance-

ment of Mechanical Science, and more particularly for promoting the acquisition of that species of knowledge which constitutes the profession of a Civil Engineer; being the art of directing the great sources of power in Nature for the use and convenience of man, as the means of production and of traffic in States both for external and internal trade, as applied in the construction of roads, bridges, aqueducts, canals, river navigation and docks, for internal intercourse and exchange; and in the construction of ports, harbours, moles, breakwaters and lighthouses, and in the art of navigation by artificial power for the purposes of commerce; and in the construction and adoption of machinery; and in the drainage of cities and towns."

This organization is very similar to our own, and its status is so well known as not to require detailed reference here. All members admitted below the grade of Member are required to pass examinations. The list contains the names of more than 9 000 members of all grades.

The Canadian Society of Civil Engineers was organized in 1887. Its objects are:

1. To facilitate the acquirement and exchange of professional knowledge among its members; and
2. To encourage original investigation.

Article 2 of the By-Laws reads as follows:

"The term 'Civil Engineer', as used in this Society, shall mean all who are, or have been, engaged in the designing or constructing of railways, canals, harbours, lighthouses, bridges, roads, river improvements or other hydraulic works, sanitary, electrical, mining, mechanical, or military works, or in the study and practice of navigation by water or air, or in the directing of the great sources of power in nature for the use and convenience of man."

It will be noted that this Society conceives civil engineering as embracing all engineering activity.

The standard of membership is not as high as in the Institution or our Society, but candidates who are not graduates of recognized technical colleges are required to pass examinations. It is divided into four sections: Electrical, Mechanical, Mining, and General. Note the absence of Civil. Provision is made for branches in important centers. They have a published code of ethics, and in two Provinces, Manitoba and Quebec, the practice of engineering is limited by law to members of the Society. Efforts are in progress to secure similar

laws in all other Provinces. The list shows 2 627 members of all grades.

Members need no description of our own organization. While our growth has been phenomenal, and the standard of our membership is generally maintained at a high level, there is a wide-spread feeling that there is something wrong, and that we are not accomplishing the good there is in us. Section 3 of Article I of our Constitution reads as follows:

"Its object shall be the advancement of engineering knowledge and practice and the maintenance of a high professional standard among its members."

Our total membership in all grades is more than 7 400. I believe there are fully 40 000 men in the United States who are eligible for various grades of membership, in about the same proportion as the list is now made up. It is hardly safe to assume that the reason they are not members is because they cannot afford to be. I think it is safe to assume that though they value the prestige of membership, they feel that the benefits are too remote. The engineer's calling generally imposes isolation, and yet they all dearly love companionship and exchange of ideas with one another. What we need and ought to have is the establishment of State organizations and of sections in all important cities. This matter has been studied by a special committee and discussed at one of our Annual Meetings, but, because we felt that our duty ended with the welfare of our own members, as provided in the Constitution, the project met with no encouragement. It is true that we permit local members in large cities to band together and call themselves Associations of Members of the Am. Soc. C. E., but they, as organizations, have no voice in our government, cannot mingle with other local organizations, and, except in very large cities where no other sections already exist, the formation of prosperous associations is impossible. The history of such local associations is by no means flattering to our pride or good management. The result of our narrow policy has been that organizations, embracing members of all the callings allied to civil engineering, have sprung up in nearly all cities of 100 000 inhabitants, and other national engineering societies have their own local sections established in many of these centers, and are co-operating with other local organizations. Some of these societies are by no means local in scope. In the inde-

pendent local organizations the membership in many instances has not been of so high a standard as our own because of the necessity for a large membership in order to meet expenses, and, like the German and Canadian Societies, all classes are taken in. Nearly all these organizations contain some of our members who have joined, sometimes as a matter of civic pride and more often because they found at home that companionship which could not reach them through the medium of our *Transactions* from New York. All but a few of these local organizations are struggling for existence, and are unable to publish their papers and discussions. Some are prosperous, and one in Boston antedates our own organization by several years. It is the duty of this Society to extend a helping hand to all who need it, and to take the initiative in an effort to amalgamate gradually all engineering organizations into one homogeneous and co-operative mass, permitting each, as far as possible, to retain its name and identity. Some sacrifice on the part of all would no doubt be necessary. Union has not been possible among religious sects, but as I have heretofore shown, engineers are of necessity managers, and, with their spirit of fair play and good sense, ought to be able to get together. Call it what you will, the American Engineer Confederation, or better still, the American Confederation of Civil Engineers.

I would not lower our standard of requirement for membership, but, by extending local societies limited financial assistance where needed, we can raise the general standard, and, by doing this, can elevate the profession at large so as to make a printed code of ethics unnecessary.

The time has come for the engineer to possess his own. The public is awake to the advantages of sound engineering advice. We have it in our power, without taking sides in political contests, to exert a beneficial influence in National, State and Municipal affairs. Some of the matters in the conduct of which we should exert a marked influence are the following:

We should seek to stop the economic waste, caused by the improvement of unworthy projects for internal waterways and harbors, the means for which are often wrung from Congress on padded and duplicated commercial statistics, the principal object being to aid Congressmen in re-election.

We should seek to have the civilian engineers in the employ of the Government given proper recognition of their services in the matter of credit, rank, salary, and retired pay.

We should see that the mad schemes for flood control of our rivers are punctured before money is expended on them, and that practicable schemes, considering the entire drainage basin as a whole, are adopted and their execution begun on a sound basis as soon as possible.

In every State and in every city we should lend our organized influence toward the securing of safe and sane building laws, free from the bias of selfish private interests and trades unions; that public moneys derived from bonds and taxes are not squandered on impossible and unnecessary projects which have not been the subject of proper engineering scrutiny; that sanitary officers are chosen for their fitness, and not because they write M. D. after their names; and that sanitary methods of water supply and sewage disposal are adopted in the rural districts. We should make use of the non-technical press in all legitimate ways, in order to bring before the minds of the public at large the importance of the engineer in civil life. Whenever we let the people know that we are organized, not only for our own improvement and protection, but for the public good, we need have no fear of not receiving proper consideration. We are wont to twit our medical friends on the fact that they often bury their mistakes, while our's stand to plague us, but we dare not compare records with them on charities and public spirit.

Before undertaking to show others what they ought to do, we should first put our own house in order. Good engineering is good management. From our financial statement it would appear that we are in a very prosperous condition, which is no doubt true, and I wish to congratulate our past management on its wisdom and foresight in this respect, but are we getting all that we can out of our present resources?

Our Society house is located in what is fast becoming the business district of New York City. The present value of the land and building is not less than \$600 000. Two large rooms are maintained as Auditorium and Assembly Room, respectively. One of these is used for the fortnightly meetings at which the attendance averages about 175. It will seat 500 persons. On the occasion of our Annual Meeting both are needed for the Smoker, but as most of the members prefer

the lower room, the 700 occupants find it uncomfortably crowded and badly ventilated. The annual ball was held this year at the Hotel Astor because the Society halls were inadequate.

There are 31 persons now employed in the building. It contains a library of 80 000 volumes, with room for 150 000, also a Reading Room which was used last year by 4 887 readers. Research work for resident and non-resident members is performed at cost by the Society's staff. Our stock of publications, costing about $25 000, is kept in the building.

Interest on the value of our property at 5% makes the rental we pay for this house about $30 000 per annum. It would seem possible and wise to issue bonds and erect a building, portions of which could be rented for a sufficient amount to pay our interest, establish a sinking fund, and give us rent free. The upper floors should be rented, preferably to engineer tenants, the street floors to merchants. The top part of the building would supply our offices, auditoriums, and perhaps a roof garden. The architectural possibilities should atone for any fancied loss of dignity. Using elevators nowadays is not considered a hardship. The amount now devoted to rent would go very far in aiding struggling local sections.

Pending the working out of these larger problems, there are minor steps which can be taken at once, which will add to the value of our membership.

The volume now called "Constitution and List of Members" should be published in the form of a year-book, containing all that it now contains, and, in addition thereto, a codification of the by-laws, which have been passed from time to time by the Board of Direction and are subject to change by it. There are other matters of general interest which might be profitably included.

The minutes of our Society are now published in the *Proceedings*. Few of the members preserve the *Proceedings* because most of the papers therein are duplicated in the *Transactions*. More important items of the *Proceedings*, including the Minutes of the Board of Direction and Annual Meetings and Conventions, should be included in the year-book, and would thereby be preserved without compelling the expense of binding and preserving volumes of *Proceedings*.

The classification which the Secretary has inaugurated should be a regular annual feature, and each member should be required to

return his calling at the same time as he does his address. Steps should be taken to provide automatically for transfer to the Associate class of all members who abandon engineering practice and devote themselves exclusively to the promotion or sale of patented articles and appliances.

The provisions for expelling members guilty of improper conduct should be made less complicated and the power be placed in the hands of the Board of Direction. The Board of Direction should be empowered, and required, to espouse the cause of any member who is the object of unjust attack. Fees should be paid to members for creditable extracts of engineering articles appearing in technical papers, and for unbiased book reviews, and the advisability of continuing the monthly notes of important engineering articles, now published in the *Proceedings,* should be investigated. There are many things in the "Constitution" that should be placed in the by-laws.

I am not prepared to suggest methods by which the ideals which I have heretofore rather vaguely outlined may be brought into realization, but I feel that whenever a thing ought to be done, a way can be found to do it, and I believe a representative committee of our Society should be appointed to study what we ought to do and how to accomplish it.

Before closing I would remind you that our next Annual Convention will probably be held on the Pacific Coast, and that, in connection with the Panama-Pacific Exposition, this Society is co-operating with four National Engineering organizations in the holding of an International Engineering Congress at this Exposition. The organization and conduct of this Congress is in the hands of a Committee on Participation, composed of the Presidents and Secretaries of each of these Societies and a local Committee.

It is expected that many foreign engineers will visit the Exposition and attend the Congress. They will come as individuals, in groups, and in organizations. A Special Committee on entertainment of these foreigners has been appointed by the five National Societies, consisting of two members from each. A meeting of these Committees has been held, and substantial progress made toward the union of all our engineering strength in the laudable purpose of properly entertaining these guests. We are going to show them that American Engineers can get together, notwithstanding their separate organiza-

tions. We are going to try, through this Committee, to organize similar committees from representatives of the five engineering societies and from local societies in all the cities at which any number of foreigners are likely to stop. These committees are going to try and secure co-operation of the civic organizations in all these cities so that our guests may feel that they are the guests of the country and not mere unattached tourists and sightseers. It seems to me that in securing the co-operation of these societies for the first time in their history, the first great step toward a unification of the civil engineers of America has been taken.

It has not been customary on occasions of this kind to refer to sad subjects, but I feel that I will be pardoned if I turn from practical things for a moment to call attention to the loss which our Society and your Board of Direction have sustained in the deaths of Mr. Emil Gerber and Past-President Alfred Noble. Mr. Gerber was a conscientious and practical member of the Board of Direction, and cheerfully gave his time to the profession. He was a member of two Special Committees, Steel Columns and Struts, and Stresses in Railroad Track. He was also a member of the Finance Committee. We shall miss him greatly. Mr. Noble was a member of the Special Committee on Valuation of Public Utilities and was Chairman of the Special Committee to Investigate Conditions of Employment of, and Compensation of, Civil Engineers. He was an Honorary Member of the Institution of Civil Engineers of Great Britain. For his great service in connection with the Isthmian Canal projects, this Nation owes him a debt of gratitude which can never be repaid. It should be our duty and pleasure to see that all our influence is directed toward securing, on the part of the Government, a fitting and enduring tribute to his memory.

I cannot close this address without giving expression to my deep appreciation of and thanks for the honor which was conferred upon me in selecting me for your President.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

Paper No. 1311

CALIFORNIA PRACTICE IN HIGHWAY CONSTRUCTION.*

BY W. C. HAMMATT, M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. ALLEN HOAR AND W. C. HAMMATT.

SYNOPSIS.

This paper describes suburban and country road conditions in California and the progress in road building in that State during the past few years. The State contains plentiful supplies of good road materials, rock, gravel, asphaltic oils, etc., and there are several cement factories.

The water-bound and oiled macadam roads are described and also concrete and asphaltic pavements; and details are given as to the good and bad features of the bases and the wearing surfaces. Some costs of asphaltic pavements are included.

With exceptional advantages in regard to road materials, California is, nevertheless, in its infancy in the study of road construction. Until about six years ago, practically no permanent road surfacing had been done outside of the municipalities. In the different counties some of the more important roads had been graveled, and a great many of the county roads had been oiled, but these measures

* Presented at the meeting of May 20th, 1914.

were taken more for the purpose of maintenance than as permanent construction work. The graveling was generally done during the rainy season in order to make the roads passable, and as the gravel was gradually worked down into the mud it was replaced by more until the depth of penetration of the surface water was reached, when the road became a good gravel highway. The oiling was generally done in dry weather, and the oil and dust formed a thin crust on the surface which was broken through by heavy traffic, and if not promptly repaired the road was soon impassable.

The southern counties of the State, which have ever been the leaders in public improvements, had expended some funds prior to the general awakening in 1908. These counties developed certain classes of cheap construction (to be described later), which, for a time, gave them an enviable reputation for roads. In 1908, however, the northern counties awakened to the need of better roads, and bond issues were passed in several of these counties for the purpose of putting permanent surfaces on their principal roads. The idea of a State highway system was developed in 1911, and, in 1912, a State Highway Commission was created and a bond issue of $18 000 000 was voted for the construction of a system of State highways.

Materials.

California is rich in many of the materials which go to make up the modern road. The Coast Range is prolific in trap rock. The Sierras are composed mainly of granite, though the foot-hills are of sandstone and shale; there are, however, in the foot-hills, a few quarries of good road rock. Gravel of a good quality for either road material or concrete, is found within easy distance of nearly every locality. There are four large companies manufacturing cement in the region of San Francisco Bay; mineral oil, with an asphaltum base, is obtained in eight different fields, distributed over about one-third of the State; and the largest refinery in the United States, producing asphaltum and asphaltic road oils, is located on San Francisco Bay.

Types of Road Construction.

Only the types of road surface used on country and suburban roads will be discussed in this paper, and each type will be taken up and described separately.

Water-Bound Macadam.—This type of pavement needs no description; it has been laid extensively in California, particularly about the centers of population. In many cases it has been laid to a finished thickness of as little as 4 in., in which case the coarsest size of rock is omitted, and the macadam is laid in two sizes only. It has been found that trap rock makes the best pavement, and that some of the softer varieties have the best cementing qualities. In fact, some of the best macadam pavements which the writer has ever seen have been laid with rock which would not pass the rattler test.

Where the sub-base has been properly prepared and drained, and the rolling has been thoroughly done, and where they have had the necessary sprinkling, macadam roads have stood up remarkably well even under automobile traffic. It is a deplorable fact, however, that the average road-builder underrates the necessity for care in the preparation of the sub-base.

Two facts are worthy of special note in the building of a macadam road: First, that a good macadam road cannot be built in rainy weather or when the sub-base is soft or wet from any other cause. To obtain proper cementation of the rock, there must be considerable resistance to the rolling. Second, the duration of the rolling cannot be specified in units of time per unit of area, as it depends on the nature of the rock and on various other conditions which make the judgment of the engineer in charge necessary.

Oiled Macadam.—This type of paving is peculiar to the localities producing an asphaltic oil. There is no fixed specification for its construction, but it is laid under many different ones, according to the whims of the engineer in charge. The name covers all classes of road surface in which oil is combined with road metal as a binder. The usual method of construction is to lay the road metal and roll the various layers in the same manner as for water-bound macadam, applying to each layer by a sprinkler a certain specified quantity of asphaltic road oil. In constructing this pavement, the main difference of opinion among road builders is in regard to the quantity of oil to be applied.

As a matter of fact, a very small proportion of the total area of oiled macadam pavement constructed has been successful. Under heavy traffic it ruts and waves very badly. It is the writer's opinion that, even under the best conditions, this pavement has little justifica-

tion. Oil has more lubricating than binding properties, and its introduction into a macadam pavement has a tendency to destroy the stability of the metal. The successful examples of pavements of this class have been either in localities where climatic conditions are such as to evaporate the more volatile parts of the oil, or where the application has been made in such a manner as to achieve the same result. Oiled macadam roads which have become so rutted as to be almost impassable, have been converted into fairly good pavements by scarifying and re-rolling, thereby aerating the oil and causing the evaporation of the lighter constituents.

Under this heading should properly come a patented pavement known as Petrolithic paving, which has been developed in the Southwest, and tried somewhat in other localities. It consists essentially of the incorporation with the natural soil of a certain definite proportion of rock and a certain quantity of oil, and the tamping of the mass into a crust, or pavement, by a roller containing spike-like projections which tamp the mixture from the bottom up. This pavement has been used with some success in localities where the soil is of a sandy rather than a clayey character, and where the summer temperature is high.

The cost of oiled macadam pavements, 6 in. in finished thickness, is about 75 cents per sq. yd., and that of Petrolithic pavements of the same depth about 60 cents.

Concrete Pavements.—Very little straight concrete pavement has been laid in California without a protective wearing surface. The writer knows of one experimental piece which has been laid with expansion joints, but it has been in use too short a time to be the source of any information at present. It was put down in two courses, the finish course being laid before the base course had set. The pavement is 6 in. thick, and cost about 90 cents per sq. yd.

In order to eke out an insufficient bond issue as far as possible, the State Highway Commission specified as the standard pavement, a 4-in. concrete base with a 1½-in. asphaltic wearing surface, and ordered that the base alone should be constructed at first, and either covered with a temporary coating of oil and screenings, or left unprotected until additional funds were available. Several hundred miles of this base have been laid, a large part of which has broken up into a series of slabs by shrinkage cracks, some of them being more

than 2 in. wide. One stretch, to the writer's knowledge, has gone to pieces by surface wear, but possibly this is due to poor concrete work which might have been prevented by better inspection. The cost of this base, inclusive of grading and what little protective coating has been used, has averaged 90 cents per sq. yd.

Asphaltic Pavements.—As might be expected from the fact that asphalt of a high grade is manufactured in California, the cities and suburban districts contain a high proportion of asphaltic pavements. The standard San Francisco pavement is 2 in. of sheet-asphalt on 6 in. of concrete. In the smaller suburban cities, this is modified to $1\frac{1}{2}$ in. of sheet-asphalt on 4 in. of concrete. In late years sheet-asphalt is being replaced by so-called asphalt macadam, under the various specifications of Warrenite, Topeka specifications, etc. The concrete base often has asphalt macadam substituted therefor, and there is now a tendency toward the macadam base for asphaltic pavements.

As opinions differ largely as to the respective merits of the various classes of bases and wearing surfaces, the writer will merely give the results of his experience and his opinions derived therefrom.

Bases.

(*a*).—The concrete base has the disadvantage of a high shrinkage in setting and a large coefficient of expansion. The former causes the base to crack into slabs in setting and the consequent destruction of its monolithic quality. The latter causes a movement after the wearing surface, and, consequently, cracks in the latter. Another fault, which is one of general practice in construction rather than in the materials of construction, is that the concrete is generally laid in such a manner that the surface is gently undulating rather than absolutely uniform. This causes the wearing surface to be of varying thickness and subject to unequal movement, as will be described later under that heading. This can be obviated, however, by trimming the concrete base with a template while laying.

This base is the one most commonly used, due to a popular prejudice in its favor. Most laymen and many engineers cannot get away from the idea that a rapid chemical reaction will produce a substance more durable and serviceable than rock combined according to the laws of gravity and stability.

FIG. 1.—RESULT OF OILING A CLAY ROAD. NOTE THE CHUCK HOLES WHEN THE SKIN IS BROKEN THROUGH.

FIG. 2.—RESULT OF OILING THE SURFACE OF A WATER-BOUND MACADAM ROAD. NOTE THE CHUCK HOLES WHEN THE SKIN IS BROKEN THROUGH.

(*b*).—The asphaltic macadam base has not given good results, for the following reasons: It has no stability, but is subject to the same faults as the too-thick asphaltic wearing surface which will be described later.

In addition, asphalt in contact with soil has a tendency to promote vegetable growth, and many pavements of this class have been heaved up and broken from root growth, where, prior to the placing of the pavement, such growth did not exist.

(*c*).—A water-bound macadam base is the best for supporting an asphaltic wearing surface, for various reasons. It has a stability due to the seating of its component parts, which is not dependent on any chemical action. It has no movement under temperature changes. It is not rigid, and, therefore, does not give the destructive reaction to impact on the surface of the asphaltic pavement, which is a feature of the concrete base. The surface is free from depressions which vary the thickness of the asphaltic wearing surface.

WEARING SURFACE.

(*a*).—The limits of efficacious thickness of the wearing surface are quite small. The wearing surface must be thick enough to be held in position by its own inertia, as it has but small cohesion to the base; and it must be so thin that the compression of rolling shall be communicated throughout its entire thickness. It is inadvisable to make the wearing surface less than 1½ in., or more than 2 in., in thickness. Where the asphaltic wearing surface is laid to such a thickness that the compression is not communicated throughout the entire depth, the compression is not uniform, and the subsequent adjustment, which always takes place in an asphaltic mixture of insufficient density, causes inequalities in the surface which are starting points for failure by waving.

(*b*).—The quality of the pavement and the roughness of the surface are not dependent to any great extent on the size of the coarse aggregate. The best and most durable mixtures are those containing the highest proportion of fine aggregate of 80 to 200 mesh. If the mixture is laid hot enough, and the compression is properly done, the fine material, or "mortar", will come to the surface and thoroughly fill all the voids in the coarse aggregate, which then merely serves to decrease the quantity of asphalt necessary. The only way of obtaining

the rough surface so much desired by some, is either to have insufficient fine material to fill the voids, or to lay the mixture too cold or with insufficient compression. In each case, the wearing surface will be open and spongy. Also, if the coarse aggregate projects above the general surface of the pavement, the stones are crushed or rolled by steel tires passing over them, leaving voids where disintegration may start.

The costs of the various types of asphaltic pavement are as follows:

2-in. sheet-asphalt, on a 4-in. concrete base.......$1.50 per sq. yd.

1½-in. Topeka specification, asphaltic wearing surface on a 4-in. concrete base..............$1.15 per sq. yd.

2-in. Topeka specification, asphaltic wearing surface on a 4-in. macadam base..............$1.10 per sq. yd.

The laying of an asphaltic wearing surface on an old macadam pavement, with no other preparation than the cleaning off of the fine material, has been done with great success. The cost of a 1½-in. Topeka specification wearing surface laid on old macadam is about 50 cents per sq. yd. All the costs given are based on labor and material costs prevailing in the bay counties about San Francisco.

Another matter worthy of note is the tendency of municipal governments in California to require street improvement work in the outlying districts to follow the same lines as in the thickly settled parts. Where the property is in large tracts, the villas have their own driveways, so that there is no occasion for teams or vehicles to stop in front of the property, nevertheless most of the municipalities require the placing of cement curbs and sidewalks, and the paving of the roadway from curb to curb, as in business blocks, thus spoiling the possibilities of architectural landscape work.

DISCUSSION

Mr. Hoar.

ALLEN HOAR, JUN. AM. SOC. C. E. (by letter).—In Mr. Hammatt's very interesting paper, he has brought out very clearly that in California there is an abundant supply of first-class road material of all kinds, well distributed over the entire State, and that, therefore, poor roads there are inexcusable. The author, however, makes several statements, which, though true, in a sense, must be misleading to those who are not thoroughly familiar with all the conditions. The writer is pleased to note that Mr. Hammatt has spoken so highly of water-bound macadam. This type of pavement when well-drained, laid on a properly prepared sub-grade, and carefully constructed, has still a great deal of usefulness.

Under the heading, "Oiled Macadam," the author says, "A very small proportion of the total area of oiled macadam pavement constructed has been successful." He also says, "even under the best conditions, this pavement has little justification." Now, as a matter of fact, as there has been some success, there is certainly some justification for its use. In the introduction to the paper Mr. Hammatt states that it has been only in the last 5 or 6 years that any real road construction has been undertaken in California, but that a great deal of graveling and oiling had been done as a temporary improvement. Such is the case, the first practice of oiling being merely to afford a temporary relief from the dust nuisance and to protect earth and gravel roads from ravages by heavy rain storms. Observation of the results obtained by this treatment led to the adoption of this material in road construction, and finally to the present perfected type of bituminous macadam. In making so broad a statement, that only a very small proportion of the oiled macadam roads has been successful, the author has presumably taken stock of all those old oiled dirt roads, only a few of which can be considered at all as oil macadam.

It is true that at first there were no fixed specifications for work of this class. It was then in the experimental stage, and each engineer was doing his own experimenting at the expense of city, county, or State. In Los Angeles County alone probably $1 500 000 were practically wasted during this experimental stage, simply because the engineer then in charge refused to profit by the experience and advice of others, but would learn for himself. Other cases of the same kind have occurred, and these unfortunate conditions have done much to blacklist pavements of this type. At present engineers engaged in work of this class have adopted similar specifications and are obtaining general success.

The author states that the successful examples of pavements of this type have been either in localities where climatic conditions are such

Mr. Hoar. as to evaporate the more volatile parts of the oil, or where the application has been made in such a manner as to obtain the same results. Now, if success is obtained by a certain method in one place, would it not be good practice to adopt such a method in other localities? As a matter of fact, an asphaltic oil which will meet the requirements of the usual specifications as now adopted has practically no lubricating properties and very little volatile matter. The foundation stones are laid and rolled until a thorough mechanical locking of the particles takes place and it is as firm and solid as a water-bound macadam pavement; then the oil, at a high temperature, is sprayed on under pressure in such a manner that it is in a finely divided state and penetrates the interstices of the stone, acting primarily as a binder. Constructed in this manner, there can be no rolling or movement of the stones, as is pointed out by Mr. Hammatt. The wearing surface of finer aggregate is then laid on this foundation.

The patented form of Petrolithic pavement is by no means an oil macadam pavement. It is merely an earth road with sand and oil incorporated to a depth of 6 or 8 in., forming a thick rubbery mat, which is pushed and pulled by the traffic stresses until it is resolved into a series of humps and hollows. In a hot climate it remains soft and spongy, and soon becomes so wavy and covered with ruts that it is almost impassable. The Petrolithic Company's tamping roller, however, has come into very good use in compacting sub-grades and earth roads. The prongs, or sheep's feet, of this roller reach through to the bottom of the sub-grade, and, starting there, compact it uniformly all the way to the surface, making a solid crust or arch to carry the weight of the finished pavement and its transmitted stresses.

Probably the strongest argument against bituminous or oil-macadam pavements is the difficulty of obtaining good inspection, which must be had to secure satisfactory results. On the contrary, an argument put forth by the advocates of concrete roads is that, because of this lack of trustworthy inspection, concrete is the only type from which satisfactory and certain results can be obtained. In practice, however, this has proven to be far from the truth. The argument is that, no matter how little care has been taken in the preparation of the sub-grade, or how poorly the concrete has been mixed, it will become hard and form a good solid foundation, and will carry the traffic safely over the bad places in the sub-grade. Experience, however, has shown that, to secure satisfactory results with concrete roads, just as much attention must be given to the inspection as for any other type; for lack of the proper quantity of cement or poor and insufficient mixing is soon indicated by rapid disintegration.

The type of wearing surface which has met with the greatest success on roads having a concrete base consists of a $\frac{3}{8}$-in. coat of oil and crushed stone screenings ranging in size from $\frac{3}{4}$ to $\frac{1}{4}$ in. The oil

is thus almost uniformly exposed to the action of the atmosphere, which causes it to "set", thus cementing the particles of the aggregate well together and to the base. As the coat is thin and uniform, it is easily applied, cheaply maintained, and permits of equal compression throughout the mass. Mr. Hoar.

W. C. HAMMATT, M. AM. SOC. C. E. (by letter).—As was expected, the writer's condemnation of oiled macadam pavements brought out some contradiction. Although this type of pavement has lost many of its advocates, there still remain many who believe in its value as a pavement. The writer, however, maintains that only a small proportion of the total mileage of oiled macadam pavement which has been laid, has been successful, and he does not include the oiled dirt and gravel roads under this heading. He knows of no road in the central part of the State, which has been under varied traffic for two years, which has remained in good condition without an expenditure of at least 8 cents per sq. yd. for repairs. There are some, where traffic is restricted to pleasure vehicles, which have remained in good condition, and these are cited as examples by advocates of this class of pavement. Mr. Hammatt.

The value of oil as a road material lies in the formation of a surface skin by the volatilization of the lighter constituents. This skin is very durable, and as long as it remains intact, it protects the general body of the pavement. In the southern part of the State, where Mr. Hoar's observations have largely been made, the sandy nature of the soil and the hot, dry climate are such as to increase the thickness of the skin. Nevertheless, in examining the roads between Los Angeles and Pasadena, the writer has noted that those carrying the automobile traffic are the only ones which hold their shape.

To sum up, the justification for the adoption of a certain type of pavement can only be from its structural excellence or its economic value. One type will not be justifiable if another type will make a better appearance and keep in better condition at a smaller cost. The writer's experience has been that a protected water-bound macadam pavement will do this. This type may be made to fit all conditions by varying the thickness of the base and of the protective covering. A macadam pavement, 6 in. thick, with a skin protection of asphalt and screenings, will cost about 70 cents per sq. yd., and will require an annual expenditure of about 9 cents per sq. yd. for upkeep. Increasing the protective covering to 2 in. in thickness of asphaltic macadam increases the cost to about $1.10 per sq. yd., and the upkeep will be nil for from 10 to 15 years. Compare these costs, as well as the appearance and usefulness of the pavement, with oiled macadam under the best conditions, and the writer believes that there will be no justification for the latter.

Mr. Hammatt. A skin of oil and screenings has been applied to some of the concrete bases of the State highway for their protection. This has been quite successful, as pointed out by Mr. Hoar, for the reason that the oil does not penetrate the concrete, and the concrete, by absorbing heat, promptly carbonizes the oil and hardens the skin. Moreover, for so thin a coating, it is probable that oil is better than asphalt, as the coating will be more resilient and cause less impact on the concrete.

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Paper No. 1312

CINDER CONCRETE FLOORS.*

By Guy B. Waite, M. Am. Soc. C. E.

With Discussion by Messrs. Arthur H. Diamant, J. R. Worcester, A. W. Buel, Charles C. Hurlbut, W. B. Claflin, F. W. Skinner, Oscar Lowinson, George E. Strehan, A. L. A. Himmelwright, Edgar Marburg, and Guy B. Waite.

Synopsis.

In the following paper the manner of using reinforced cinder concrete floor slabs by various cities in the eastern part of the United States is described. Especial attention is called to the arbitrary manner of using this material, and the insistence of continuing this arbitrary use, by New York City, as indicated by the latest building codes.

Attention is called to the unit stresses adopted by various cities and to recent data tending to establish unit working stresses from actual tests. The qualifications of this material for the purposes described are discussed, and a comparison with stone concrete is made.

The manner of arbitrary testing is described, and the difference between the conditions of such tests and the actual loadings found in buildings is pointed out.

Analysis is made of the arbitrarily approved reinforced cinder concrete floor slabs, showing the stresses which would be imposed on the materials by the maximum approved loads.

Cinder concrete is described and tabulations are made for the location of the neutral axis and for the carrying capacities, assuming

* Presented at the meeting of May 6th, 1914.

various unit stresses. The influence of the stresses due to an excess of steel or to an excess of concrete is discussed.

Analyses of stresses allowed by such tests indicate that in some cases approvals have been given for dangerous construction; it is pointed out that arbitrary tests can be made which indicate carrying capacities not possible under the conditions actually found in buildings; and it is shown by formulations and tabulations that reinforced cinder concrete may be easily and uniformly designed by adopting working unit stresses, and that several cities have adopted such stresses for this material.

The writer believes that in this paper he has amply demonstrated that the only manner in which this material can ever be uniformly used under various conditions is for each locality to adopt unit stresses suited to its own conditions.

Where cinders of good quality can be obtained, cinder concrete, reinforced with steel, for fire-proof floor construction, is used very extensively in the eastern part of the United States. It is estimated that more than 50% of the large fire-proof constructions of Philadelphia, Pa., have reinforced cinder concrete floor slabs, and New York City has probably a larger proportion.

Floor slabs of stone and gravel concrete, reinforced with steel, of the same general form as cinder concrete, have been systematized by engineers so that they are being used on a rational basis; but cinder concrete has been allowed to straggle along, without much attention from engineers in general, and is being used in a varied manner. Most large cities, such as Philadelphia, Pa., Boston, Mass., and Chicago, Ill., require cinder concrete for floor slabs to be designed, in thickness and in quantity of reinforcement, with certain values of unit stresses as a limit.

In New York City, and in most small cities of the eastern part of the United States, unit stresses are not considered in the use of such concrete. In these places, the use is either entirely arbitrary or a separate test is required for each condition.

In New York City, since the beginning of the use of cinder concrete floors, in 1896, there have been some 200 approvals for reinforced floor slabs of this material, based on load tests. These approvals were

for varying spans, thicknesses of concrete, and quantities of reinforcement. The tests have generally been conducted with a uniformly distributed load—of pig iron, bags of cement, or sand—imposed on the slab.

The writer has been informed that since 1911 concentrated load tests are required for all new approvals, by the Borough of Manhattan, New York City. As will be shown in detail, the results of former tests, which include all principal constructions in use, are extremely variable. The result is that now there are approvals of concrete floor slabs in which the effective depth is 3 in. (4 in. total thickness) and the reinforcement is less than ½ lb. per sq. ft. on an 8-ft. span, and with a live floor load of 250 lb. per sq. ft. In these slabs the concrete is in the proportion of 1 : 2 : 5.

One of the dealers in reinforcing materials for cinder concrete publishes a statement of what he calls its "authorized use" by the Boroughs of Greater New York City. This is a 1 : 2 : 4 mixture, with a 5-in. effective depth up to 12-ft. spans for live loads of 60 lb. per sq. ft.—or a total load of about 120 lb.

Each contractor in New York City is held strictly to the details of construction for his approvals. For instance, if the approval allows a live load of 250 lb. per sq. ft. up to 6-ft. spans, and it is desired to use the construction for a live load of only 60 lb. per sq. ft., on 6 ft. 6-in. spans, this construction will not be allowed for this excess of 6 in. in span, though stressed less than with the approved loads.

Moreover, stone and gravel reinforced concrete floor slabs are required to be designed according to the following values: Extreme fiber stress in concrete, 650 lb. per sq. in.; maximum tension in mild steel, 16 000 lb. per sq. in.; ratio of moduli, 15. According to these values, stone concrete slabs in some instances would be designed with a greater thickness and have greater reinforcement than the approvals for cinder concrete.

The writer believes that the excuse, that cinder concrete is so extremely variable as not to permit of the assignment of unit stresses, is not well founded. The fact that cinder concrete is being used extensively in engineering construction is sufficient reason for assigning, for safety, some unit stresses, in order to insure uniformity. The present manner of making arbitrary approvals has led to confusion, various forms of reinforcement obtaining approvals in which the

spans, thicknesses of concrete, quantities of reinforcement, and floor loads vary beyond description.

The proposed building codes of 1912 and 1913 have persistently continued the present manner of using cinder concrete (with slight modifications) for floor constructions.* In these codes, the manner of designing and constructing cinder concrete floor slabs is arbitrarily specified. In the proposed code of 1913, the allowed carrying capacity of the slab with 3 in. effective depth on 8-ft. spans is increased (over the 1912 code) to a live load of 250 lb. per sq. ft.

It is on account of these conditions in the uses of cinder concrete that the writer has been prompted to bring this matter before the Society, with the hope of interesting the Engineering Profession in a subject which should be of vital concern to many.

Cinders.—Cinders for concrete floor construction are understood to mean either hard or soft coal ashes coming from boiler plants. Coal ashes from a stove or a small house boiler are generally too finely burned to be of use in this kind of concrete. Cinders from either hard or soft coal, and having sharp particles greatly in excess of the smooth ashes, make a concrete of considerable strength and of good fire-resisting qualities.

Cinders vary in size, from the consistency of coarse building sand to clinkers several inches in diameter. Some hard-coal cinders run so perfectly in grade that good concrete is made without the addition of sand; more often, however, sand must be added. The quantity of sand required to fill the voids is variable, but the usual specification is 2 parts of sand to 5 parts of cinders.

After a long experience in the use of cinder concrete, the writer believes that there should be no danger in assigning working stresses to the material, these stresses being taken from tests on known mixtures in which cinders of the lowest grade permitted are used. If the material is too poor to have such working stresses assigned, it seems evident that cinder concrete is not a safe material for such an important part of the structure as the floor slab.

Cinder Concrete as Fire-Proofing.—Since 1896 some 82 different floor constructions have been tested in the United States by fire and water. Nearly all these tests were made under the auspices of the Bureau of Buildings of New York City, on full-sized floor slabs. About

* Sec. 113, proposed code of 1912; and Sec. 104, proposed code of 1913.

40 of the tests were on reinforced cinder concrete floor slabs, some 23 on stone or gravel concrete floor slabs, 10 on some form of hollow tile, and the others on special floor constructions.

The test structures were about 14 ft. square and 9 ft. high, with a fire grate at the bottom and the floor slabs to be tested forming the ceiling. The average temperature on the tested floor slabs was 1 700° Fahr. maintained for 4 hours. Following this there was a water test of 60 lb. pressure through a 1⅛-in. nozzle applied for 10 min. to the under side of the slabs. During the fire test a load of 150 lb. per sq. ft. remained on the entire surface of the slabs. After the fire test the slabs were subjected to a total load of 600 lb. per. sq. ft. Fig. 1 is a view inside one of these test structures after a fire test.

In these tests, the cinder concrete floor constructions withstood the fire and water better than any other material. In the writer's opinion, the reason for this is not due to cinder concrete being more fire-proof than the others, but simply to the fact that it is not ruptured and destroyed by the expansions and contractions caused by fire and water.

Hollow tile, as a material, may be more fire-proof than cinder concrete, but the stresses due to the expansion of the exposed surface, destroy it.

The test structure, an inside view of which is shown, was built entirely of cinder concrete, mixed 1 to 4 without sand, and withstood four different fires before being torn down to make room for improvements. Only the surface, for a depth of less than 1 in., was affected; this was dehydrated, but the burned cinder concrete was still intact, and protected the remainder of the material. In this concrete there was at least 5% of unburned pea coal which was unaffected by the fire.

Arbitrary Testing of Cinder Concrete.—In New York City after each type of cinder concrete construction had qualified for fire-proofing, the variations in spans, carrying capacity, etc., were determined by each manufacturer by constructing the slabs he proposed to use and testing them with uniformly distributed loads under the supervision of the Bureau of Buildings.

This was the method used for the approvals of the constructions shown, with the exception of Fig. 5, which was constructed for fire test. These constructions were taken from the files of the Bureau of Buildings from among some 200 similar approvals.

For comparison with the approved cinder concrete floor slabs shown by Figs. 3 to 7, the writer gives the relative thicknesses and the quantity of reinforcement for slabs under similar conditions according to the unit stresses used by Philadelphia, and also the thicknesses and quantity of reinforcement required by New York City for 1:2:4 stone concrete.

It is to be noted, in comparing the stone or gravel concrete construction (Figs. 11 to 13) with the cinder concrete, that the former is made unquestionably continuous, and the latter, in most cases, is cut into and is simply supported by the steel beams (Figs. 3 to 10, and 14 and 15).

The approvals for the longest spans with the highest carrying capacity were based on tests with slabs which were continuous over supports (Figs. 3 to 7). In practice, it is seldom that cinder concrete floor slabs between steel beams can be made continuous throughout a floor.

In Figs. 14 and 15, which are assumptions for illustration, it will be noticed that the reinforcement is simply laid over the top flanges of the supporting steel beams. In practice, there is no way of taking up the slack in the reinforcement coming over the tops of the beams, and the consequences are that under such conditions there can be no continuous action—such as engineers understand—in the combined steel and concrete structure. Unless considerable slack is allowed on the sides of each supporting beam, the tamping of the concrete works the reinforcement to the top of the slab.

In buildings where a wooden floor is used, the sleepers are generally laid directly on the steel beams, and consequently the concrete is kept below the top flanges of the beams. Here, therefore, the concrete is not continuous in any part of the floor; but in the interior spaces on all floors, whether continuous or not, the channels on the outside spans, and against openings in floors, cannot have either concrete or reinforcement continuous over them (Figs. 14 and 15).

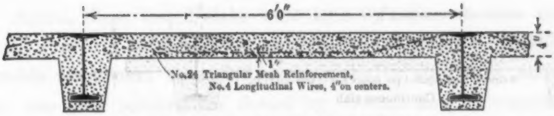
To any one acquainted with the actual construction of cinder concrete floors, it must be apparent that some parts of each floor will not admit of being constructed with the same care that might be attained in a test construction; therefore it does not follow that because a slab was continuous in the test, it is so in actual practice.

FIG. 1.—CEILING AND SIDE OF CINDER CONCRETE, AFTER THE SECOND FIRE. PLASTER ON PART OF CEILING.

FIG. 2.—ARBITRARY TEST LOAD. SAND IN BAGS ON 4-IN. SLAB, 36 IN. WIDE AND 6 FT. LONG.

Fig. 1.

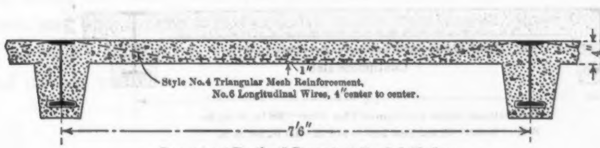
Fig. 2.—Arbitrary Test Load.



Concrete: 1 Portland Cement, 2 Sand, 5 Cinders.

Working Load: 400 lb. per Sq. Ft.

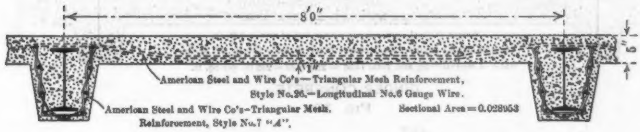
FIG. 3.



Concrete: 1 Portland Cement, 2 Sand, 5 Cinders.

Working Load: 95 lb. per Sq. Ft.

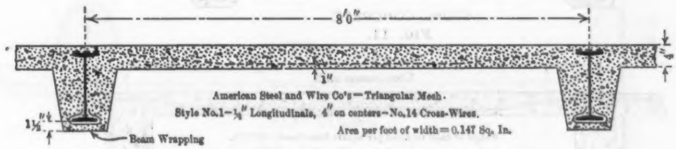
FIG. 4.



Concrete: 1 Portland Cement, 2 Sand, 5 Cinders.

Working Load: 150 lb. per Sq. Ft.

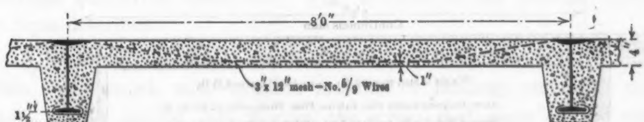
FIG. 5.



Concrete: 1 Portland Cement, 2 Sand, 5 Cinders.

Working Load: 200 lb. per Sq. Ft.

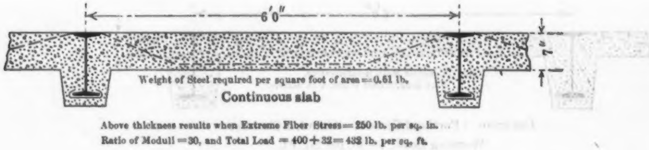
FIG. 6.



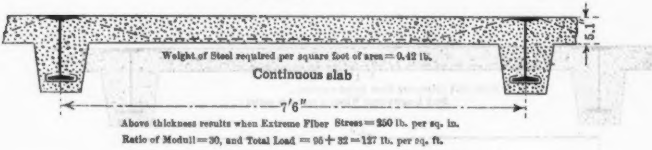
Concrete: 1 Portland Cement, 2 Sand, 5 Cinders.

Working Load: 250 lb. per Sq. Ft.

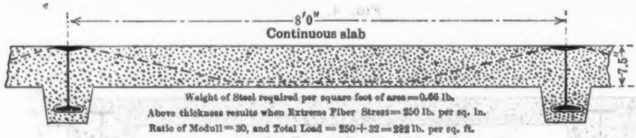
FIG. 7.



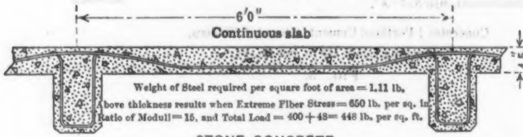
CINDER CONCRETE.
FIG. 8.



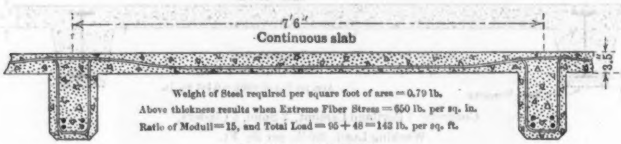
CINDER CONCRETE.
FIG. 9.



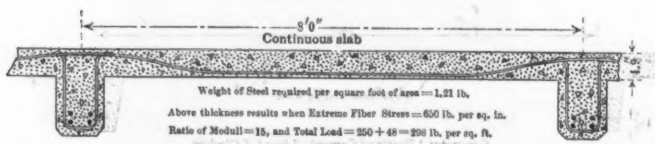
CINDER CONCRETE.
FIG. 10.



STONE CONCRETE.
FIG. 11.



STONE CONCRETE
FIG. 12.



STONE CONCRETE.
FIG. 13.

Again, these tested slabs were held by adjacent slabs on each side of the supports, thus enabling each side to take thrust. Now, in practice, outside slabs next to walls, elevator shafts, stairways, show windows, etc., are only resisted for thrust by tie-rods and by concrete slabs on one side.

If the writer is correct in his statement that outside slabs, etc., are not under the same conditions as the tested ones, where the reinforcement and the concrete is carefully made continuous, then this form of so-called continuous construction is being used in an unapproved and unsafe manner.

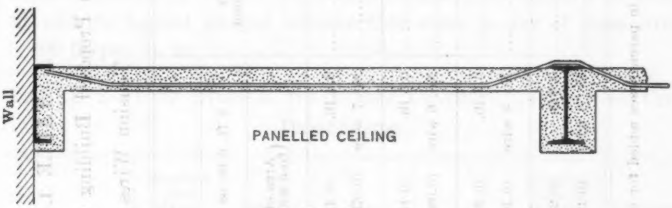


FIG. 14.

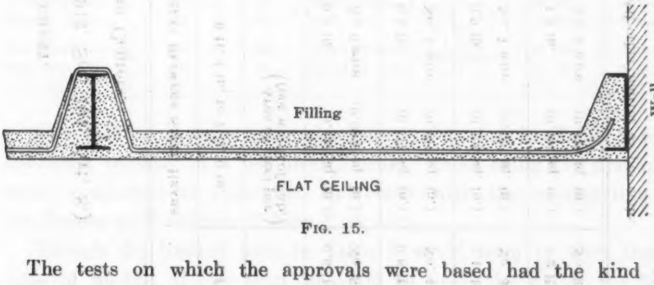


FIG. 15.

The tests on which the approvals were based had the kind of loadings heretofore mentioned. Fig. 2 shows a test loading similar to those described.

With the conditions of continuous construction under which these slabs were tested, with possible arching of loading, and with supports braced against side flexure, the stress that came on the steel and concrete at the middle of the slab can only be guessed. Still, according to the latest proposed building code (that of 1913) tests and

TABLE 1.—REINFORCEMENT.

(From Proposed Building Code of 1913, Sec. 104, Art. 8.)

Tension Wires 4 Inches on Centers.

Live load per square foot of floor area.	Type of reinforcement.	Span Between Steel Beams. 6 ft. 0 in. or less.	(Area of steel per foot width of slab.)	6 ft. 1 in. to 7 ft. 0 in.	(Area of steel per foot width of slab.)	7 ft. 1 in. to 8 ft. 0 in.	(Area of steel per foot width of slab.)
100 lb. and less	Bars	0.4 lb.	(0.118 sq in.)	0.5 lb.	(0.147 sq. in.)	0.6 lb.	(0.177 sq. in.)
	Wire mesh	No. 7 wire	(0.0735 sq. in.)	No. 6 wire	(0.0868 sq. in.)	No. 5 wire	(0.101 sq. in.)
101 to 150 lb.	Bars	0.5 lb.	(0.147 sq. in.)	0.6 lb.	(0.177 sq. in.)	0.8 lb.	(0.236 sq. in.)
	Wire mesh	No. 6 wire	(0.0868 sq. in.)	No. 5 wire	(0.101 sq. in.)	No. 4 wire	(0.120 sq. in.)
151 to 200 lb.	Bars	0.7 lb.	(0.206 sq. in.)	0.9 lb.	(0.265 sq. in.)	1.1 lb.	(0.324 sq. in.)
	Wire mesh	No. 5 wire	(0.101 sq. in.)	No. 4 wire	(0.120 sq. in.)	No. 3 wire	(0.140 sq. in.)
201 to 250 lb.	Bars	0.8 lb.	(0.236 sq. in.)	1.0 lb.	(0.294 sq. in.)	1.2 lb.	(0.353 sq. in.)
	Wire mesh	No. 4 wire	(0.120 sq. in.)	No. 3 wire	(0.140 sq. in.)	No. 2 wire	(0.162 sq. in.)

Matter in parentheses added for comparison. All other matter as per code.

approvals of that kind were to have preference over unit stresses. The following is quoted from Sec. 104, Art. 8, of this proposed code:

"The concrete is made 1, 2, 5; slab thickness, 4 in., with reinforcement 1 in. from bottom.

"Reinforcement must be as per tabulations."

It will be noticed from Table 1 that when the reinforcement is of wire mesh the area required is only about one-half as great as for any other form of reinforcement—even with the same kind of wire but without the mesh.

Use of Cinder Concrete by Cities Other than New York.—The cities mentioned in Table 2 use a ratio of moduli of elasticity of about 30, and the highest allowed extreme fiber stress in any of these cities is 300 lb. per sq. in.

TABLE 2.—UNIT STRESSES FOR CINDER CONCRETE, AS USED BY CITY DEPARTMENTS.

City.	Extreme fiber stress, in pounds per square inch.	Ratio of moduli of elasticity.	Kind of cinders used.	Mixture.	Working stresses in steel, in pounds per square inch.	Preferred reinforcement.
Philadelphia, Pa.	250	30	Hard coal...	1 : 2 : 4	16 000	None.
Boston, Mass....	300	25	Soft coal....	1 : 2 : 4 to 1 : 2½ : 5	16 000	None.
Chicago, Ill......	245	30	Hard or soft.	1 to 8 aggregate.	18 000	None.
Baltimore, Md...	300	30	Hard or soft.	1 : 2 : 4.	15 000	None.

The results of tests on cinder concrete quoted in Table 3 were published recently in a technical journal,* these being the first of a series conducted at Columbia University with the co-operation of the Bureau of Buildings of New York City.

Though the limited tests in Table 3 would seem to show that a ratio of moduli of less than 30 could be used, the writer has been informed that specimens "C" were from extra good concrete, and that "A" and "B" are nearer the average material. In this case the ratios used by Philadelphia, Chicago, and Baltimore seem to be about right. As will be seen later, a less ratio of moduli will give a less carrying capacity, other things remaining the same.

**Engineering News*, October 9th, 1913.

In the tests at Columbia University, just referred to, it is the feeling, at the present stage of these tests, that an extreme fiber stress of from 200 to 250 lb. per sq. in. on cinder concrete is all that can be used conservatively.

Only the tests up to 6 months old have been made; the final tests may alter this assumption, but, for discussion, these values are sufficiently correct.

TABLE 3.

"WEIGHT AND COMPRESSIVE STRENGTH OF CINDER CONCRETE AS USED IN FIREPROOF FLOORS, NEW YORK CITY.

(The table covers 120 samples. Each figure given is the average of ten samples.)

Zone.	A.	B.	B2.	C.
Mix	1:2:5	1:1:5	1:2:5	1:2:5
Weight, lb. per cu. ft.	107	100	107	109
One-month Test:				
Crushing Strength, lb. per sq. in.	407	507	818	980
Mod. of Elast., lb. per sq. in.	924 600	857 400	1 230 000	1 492 000
Two-month Test:				
Crushing Strength, lb. per sq. in.	701	662	1 254	1 035
Mod. of Elast., lb. per sq. in.	1 134 000	1 030 000	1 740 000	1 428 250
Six-month test:				
Crushing Strength, lb. per sq. in.	933	754	1 744	1 478
Mod. of Elast., lb. per sq. in.	971 000	1 050 000	1 348 000	1 276 000

NOTE.—B was hand-mixed; A, B2, and C were machine-mixed. The modulus of elasticity was determined at a point of the elastic curve corresponding to one-fourth the ultimate strength."

Designing Reinforced Concrete Floor Slabs.—Solid slabs of cinder concrete are not weak in shear, so that the discussion of vertical and diagonal shear will not be necessary. In the writer's observation of many breaking tests on cinder concrete floor slabs, there has never been a failure near the supports; almost uniformly, the breaking has taken place near the center of the slab. Usually, the top of the central portion has spalled off (cup fashion), apparently due to horizontal shear and compression, after which the slab suddenly collapsed.

Adhesion.—The adhesion of concrete to various kinds of reinforcement is shown by Table 4, the results of a series of tests on 1:2:4 cinder concrete, made in 1904 by Messrs. H. B. Gaylord and H. A. Pratt, at Stevens Institute of Technology. The cement was Lehigh Portland, and the cinders were as found coming from a boiler plant. The concrete blocks were 6 by 6 in., and the steel was embedded in the center.

It may be noted from Table 4 that the plain bars held in adhesion quite as well as the deformed bars, the average adhesion of surface embedded being about 200 lb. per sq. in. This same fact, as to the relative adhesion of plain and deformed steel in stone concrete, has already been shown in numerous tests. Therefore, it will be considered that the form of the reinforcement is immaterial so long as it has sufficient surface embedded.

Bending.—Proceeding on the ordinary assumptions of the common theory of flexure, Fig. 16 is a stress diagram, at the right of which

TABLE 4.—RESULTS OF ADHESION TESTS ON 1:2:4 CINDER CONCRETE.

No. of specimen.	Length embedded, in inches.	Perimeter, in, inches.	Area of cross-section, in square inches.	Shape of bar.	Kind of concrete.	Pull, in pounds per inch.	Pull, per inch of perimeter.	Pull per square inch.
1	6¼	1.75	0.094	De Mann.	Cinder, 1:2:4:	396	1 414	206.2
2	11⅜	1.75	0.094	"	"	598	3 885	247.2
3	6¼	1.75	0.094	Flat.	"	376	1 343	214.6
4	16	1.75	0.094	"	"	250	2 285	142.8
5	18	1.75	0.094	"	"	230	2 371	131.4
6	23	1.75	0.094	"	"	176	2 072	89.8
7	5	1.57	0.196	Round.	"	400	2 730	255.0
8	6⅛	1.57	0.196	"	"	553	2 076	339.2
9	9	1.57	0.196	"	"	647	3 710	412.2
10	13	1.57	0.196	"	"	379	5 460	196.0
11	16	1.57	0.196	"	"	417	4 331	277.0
12	28¾	1.57	0.196	"	"	239	4 188	145.0
13	6	1.625	0.156	⅛ by $\frac{5}{16}$-in.	"	500	1 834	307.6
14	10¼	1.625	0.156	"	"	146	914	90.0
15	12	1.625	0.156	"	"	500	3 668	308.0
16	17¼	1.625	0.156	"	"	345	3 661	212.3
17	19¾	1.625	0.156	"	"	310	3 778	191.0
18	27¼	1.625	0.156	"	"	232	3 895	143.0
19	4¾	3.25	0.625	1 by ⅝-in.	"	990	1 477	311.0
20	6¾	3.25	0.625	"	"	800	1 661	246.3
21	14¾	3.25	0.625	"	"	665	3 015	204.6
22	20¼	3.25	0.625	"	"	680	4 353	215.0
23	26¾	3.25	0.625	"	"	645	5 303	193.0
24	11⅛	3.125	0.203	1 by ⅜-in. channel.	Stone.	895	3 168	247.9
25	18	3.125	0.203	"	"	610	3 505	194.7
26	22½	3.125	0.203	"	"	523	3 433	103.8
27	6⅜	3.125	0.203	"	Cinder.	840	1 715	273.1
28	8⅛	3.125	0.203	"	"	848	2 304	271.1
29	16¾	3.125	0.203	"	"	245	1 120	166.8
30	17	3.125	0.203	"	"	555	3 019	177.6
31	20¼	3.125	0.203	"	"	435	2 216	141.0
32	22½	3.125	0.203	"	"	476	3 435	154.0
33	35¾	3.125	0.203	"	"	294	3 357	93.9
34	6½	1.00	0.0625	Ransome.	"	431	2 800	439.8
35	7½	1.00	0.0625	"	"	377	2 825	376.6
36	10	1.00	0.0625	"	"	409	4 085	408.5
37	13¼	1.00	0.0625	"	"	334	4 435	334.8
38	22	1.00	0.0625	"	"	270	5 250	238.6
39	26½	1.00	0.0625	"	"	755	2 000	75.4
40	6	1.00	0.0625	Square.	"	442	2 650	441.6
41	7¾	1.00	0.0625	"	"	322	2 500	332.5
42	11½	1.00	0.0625	"	"	322	2 500	217.4
43	15½	1.00	0.0625	"	"	261	3 200	206.4
44	25¼	1.00	0.0625	"	"	128	3 215	127.3
45	32¾	1.00	0.0625	"	"	113	3 650	111.1

is shown the strain diagram used in determining the location of the neutral axis. Fig. 17 is for illustration, and Fig. 18 is a heavy-load stress diagram for purposes of discussion.

Let E_c = the modulus of elasticity of cinder concrete;
" E_s = " " " " " mild steel;
" f_c = " extreme fiber stress on cinder concrete (working);
" f_s = " unit allowed stresses per square inch on mild steel;
" a = " area of reinforcing steel, in square inches, or in pounds per square foot;
" y_1 = " distance from top of slab to neutral axis, or
" y_2 = " " " neutral axis to center of gravity of steel;
" d = " distance from center of gravity of steel to top of slab;
" T = " total thickness of the slab; here taken as $d + 1$.

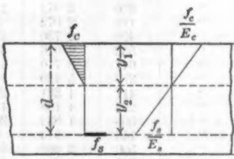


FIG. 16.

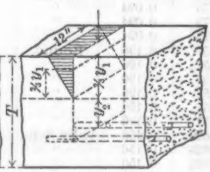


FIG. 17.

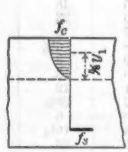


FIG. 18.

Referring to the diagram at the right of Fig. 16, it is evident that the maximum strain on the extreme fibers is $\frac{f_c}{E_c}$ and that the strain on the steel is $\frac{f_s}{E_s}$. Both these strains being the bases of two similar triangles, they are proportional to their altitudes, or $\frac{f_c}{E_c} : \frac{f_s}{E_s} = y_1 : y_2$; whence, $y_2 = y_1 \frac{f_s}{f_c} \times \frac{E_c}{E_s}$. If, $\frac{f_s}{f_c} = n$, and $\frac{E_c}{E_s} = m$, then $y_2 = n\,m\,y_1$; but, $d = y_2 + y_1$; hence,

$$y_1 = \frac{d}{n\,m + 1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Tabulation of Values for y_1.—As there may be differences of opinion as to the proper ratio of the moduli of cinder concrete and steel, the writer has prepared Table 5, giving values for y_1 (the dis-

tance from the top of the slab to the neutral axis) for different moduli, and for different ratios of unit stresses of extreme fiber on the concrete, and tension in the steel.

TABLE 5.—DISTANCE FROM EXTREME FIBER OF CONCRETE TO NEUTRAL AXIS, IN TERMS OF d.

$\frac{E_s}{E_c}$ where E_s = 30 000 000 and E_c ranges from 800 000 to 2 500 000	y_1 when $\frac{f_s}{f_c} = 80$ i. e., $\frac{16\,000}{200}$ or $\frac{20\,000}{250}$	y_1 when $\frac{f_s}{f_c} = 64$ i. e., $\frac{16\,000}{250}$ or $\frac{20\,000}{312.5}$	y_1 when $\frac{f_s}{f_c} = 53.3$ i. e., $\frac{16\,000}{300}$ or $\frac{20\,000}{375}$	y_1 when $\frac{f_s}{f_c} = 49.3$ i. e., $\frac{16\,000}{325}$ or $\frac{20\,000}{406}$	y_1 when $\frac{f_s}{f_c} = 32$ i. e., $\frac{16\,000}{500}$ or $\frac{20\,000}{625}$	y_1 when $\frac{f_s}{f_c} = 24.6$ i. e., $\frac{16\,000}{650}$ or $\frac{20\,000}{810}$
37.5	0.32d	0.37d	0.41d	0.43d	0.54d	0.60d
33.3	0.29d	0.34d	0.39d	0.40d	0.51d	0.57d
30.0	0.27d	0.32d	0.36d	0.38d	0.48d	0.55d
27.3	0.26d	0.30d	0.34d	0.36d	0.46d	0.52d
25.0	0.24d	0.28d	0.32d	0.34d	0.44d	0.50d
23.1	0.22d	0.27d	0.30d	0.32d	0.42d	0.48d
21.4	0.21d	0.25d	0.29d	0.30d	0.40d	0.47d
20.0	0.20d	0.24d	0.27d	0.29d	0.38d	0.45d
18.8	0.19d	0.23d	0.26d	0.28d	0.37d	0.43d
17.6	0.18d	0.22d	0.25d	0.26d	0.36d	0.42d
16.7	0.17d	0.21d	0.24d	0.25d	0.34d	0.40d
15.8	0.17d	0.20d	0.23d	0.24d	0.33d	0.39d
15.0	0.16d	0.19d	0.22d	0.23d	0.32d	0.38d
14.3	0.15d	0.18d	0.21d	0.23d	0.31d	0.37d
13.6	0.15d	0.18d	0.20d	0.22d	0.30d	0.36d
13.0	0.14d	0.17d	0.20d	0.21d	0.29d	0.35d
12.5	0.14d	0.16d	0.19d	0.20d	0.28d	0.34d
12.0	0.13d	0.16d	0.19d	0.20d	0.27d	0.33d

In arriving at the ratio of moduli, it was assumed that the modulus of elasticity of mild steel was about 30 000 000, and that the modulus of elasticity of cinder concrete varied from about 800 000 to 2 500 000. y_1 is given for this range of moduli and for extreme fiber stresses on the concrete of 200, 250, 300, 500, and 650 lb. per sq. in. Thus, it will be found that Table 5 will cover values of y_1 for the ordinary assumptions for stone concrete, as well as for cinder concrete.

Thickness of Concrete.—Having the values of y_1, the carrying capacity of reinforced cinder concrete slabs for varying spans and varying loads can be tabulated.

Let W = total floor load, in pounds per square foot;
" L = span of slab between supports, in feet;
" M = bending moment from loading;
" R = resisting moment of slab.

Then, the bending moment (for simply supported slabs), in inch-pounds, coming on the slab is,

$$M = \frac{12\,W L^2}{8},$$

the resisting moment (Figs. 16 and 17) is:

$$R = M = 12 \times \frac{y_1}{2} f_c \left(\frac{2}{3} y_1 + y_2\right),$$

$$\text{hence, } \frac{W L^2}{4} = y_1 f_c \left(\frac{2}{3} y_1 + y_2\right) \quad (2)$$

For the outside spans of continuous slabs:

$$\frac{W L^2}{5} = y_1 f_c \left(\frac{2}{3} y_1 + y_2\right) \quad (3)$$

For slabs continuous throughout:

$$\frac{W L^2}{6} = y_1 f_c \left(\frac{2}{3} y_1 + y_2\right) \quad (4)$$

If $\frac{E_s}{E_c} = 30$, that is, $\frac{30\,000\,000}{1\,000\,000}$, and $\frac{f_s}{f_c} = 80$, that is, $\frac{16\,000}{200}$, then, for slabs simply supported (see values of y_1 and y_2 in Table 5):

$$\frac{W L^2}{8} = 24.57\, d^2, \text{ and}$$

$$d = 0.0713\, L \sqrt{W} \quad (5)$$

$$T = d + 1 \quad (6)$$

Equations 5 and 6 are for the conditions shown in Table 8.

The values for d (and for T), the thickness of concrete slabs given in other tables, were derived by substituting the proper values of y_1 and y_2, in terms of d, in Equations 2 and 4, and reducing to Equations 5 and 6.

Of course, the thicknesses of the slabs, quantities of reinforcement, and carrying capacities are based on the relative quantities of concrete and steel. An excess of concrete or an excess of steel will alter the position of the neutral axis (Equation 1) and the carrying capacity. In these calculations the working values of f_c, etc., are about one-quarter of the ultimate; that is, at a point in the stress-strain

TABLE 6.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\ 000}{300} = 53.3$; $\frac{E_s}{E_c} = 30$; $y_1 = 0.36\ d$; $y_2 = 0.64\ d$, FROM TABLE 5.

Span, in feet.	Total Uniform Load, in Pounds per Square Foot. 120		150		175		200		225		250		300		350	
	T, Total thickness of slab, in inches.	*W*, Quantity of steel, in pounds per square foot of area.	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*
5.0	3.8	0.39	4.1	0.43	4.4	0.47	4.6	0.50	4.8	0.52	5.1	0.55	5.4	0.61	5.8	0.66
5.5	4.1	0.43	4.5	0.48	4.7	0.51	5.0	0.55	5.2	0.58	5.5	0.62	5.9	0.68	6.3	0.73
6.0	4.4	0.47	4.8	0.52	5.1	0.55	5.4	0.61	5.6	0.63	5.9	0.68	6.3	0.73	6.8	0.80
6.5	4.7	0.51	5.1	0.57	5.4	0.61	5.7	0.65	6.0	0.69	6.3	0.73	6.8	0.80	7.2	0.86
7.0	4.9	0.54	5.4	0.61	5.7	0.65	6.1	0.70	6.4	0.75	6.7	0.79	7.2	0.86	7.7	0.92
7.5	5.2	0.58	5.7	0.65	6.1	0.70	6.4	0.75	6.8	0.80	7.1	0.84	7.7	0.92	8.2	0.99
8.0	5.5	0.62	6.0	0.69	6.4	0.74	6.8	0.80	7.2	0.86	7.5	0.90	8.1	0.98	8.7	1.06

$d = 0.0518\ l \sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.138\ d$.

TABLE 7.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\ 000}{250} = 64$; $\frac{E_s}{E_c} = 30$; $y_1 = 0.32\ d$; $y_2 = 0.68\ d$, FROM TABLE 5.

Span, in feet.	Total Uniform Load, in Pounds per Square Foot. 120		150		175		200		225		250		300	
	T, Total thickness of slab, in inches.	*W*, Quantity of steel, in pounds per square foot of area.	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*
5.0	4.2	0.33	4.6	0.37	4.9	0.40	5.2	0.43	5.4	0.45	5.7	0.48	6.1	0.52
5.5	4.6	0.37	5.0	0.41	5.3	0.44	5.6	0.47	5.9	0.50	6.1	0.52	6.6	0.57
6.0	4.9	0.40	5.3	0.44	5.7	0.48	6.0	0.51	6.3	0.54	6.6	0.57	7.1	0.62
6.5	5.2	0.43	5.6	0.47	6.1	0.52	6.4	0.55	6.7	0.58	7.1	0.62	7.6	0.67
7.0	5.5	0.46	6.1	0.52	6.5	0.56	6.8	0.59	7.2	0.63	7.5	0.66	8.1	0.72
7.5	5.8	0.49	6.4	0.55	6.9	0.60	7.3	0.64	7.6	0.67	8.0	0.71	8.7	0.79
8.0	6.2	0.53	6.8	0.59	7.2	0.63	7.7	0.68	8.1	0.72	8.5	0.76	9.2	0.84

$d = 0.059\ l \sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.102\ d$.

TABLE 8.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{200} = 80$; $\frac{E_s}{E_c} = 30$; $y_1 = 0.27\,d$; $y_2 = 0.73\,d$, FROM TABLE 5.

Span, in feet.	Total Uniform Load, in Pounds per Square Foot. 120		150		175		200		225	
	T, Total thickness of slab, in inches.	W, Quantity of steel, in pounds per square foot of area.	T.	W.	T.	W.	T.	W.	T.	W.
5.0	4.9	0.27	5.4	0.30	5.7	0.32	6.0	0.34	6.3	0.36
5.5	5.3	0.29	5.8	0.33	6.2	0.36	6.5	0.38	6.9	0.41
6.0	5.7	0.32	6.2	0.36	6.7	0.39	7.0	0.41	7.4	0.44
6.5	6.1	0.35	6.7	0.39	7.1	0.42	7.5	0.45	7.9	0.47
7.0	6.5	0.38	7.1	0.42	7.6	0.45	8.1	0.49	8.5	0.52
7.5	6.9	0.40	7.5	0.45	8.1	0.49	8.6	0.52	9.0	0.55
8.0	7.2	0.43	8.0	0.48	8.5	0.52	9.1	0.56	9.5	0.58

$d = 0.0713\,l\,\sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.068\,d$.

TABLE 9.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{300} = 53.3$; $\frac{E_s}{E_c} = 30$; $y_1 = 0.36\,d$; $y_2 = 0.64\,d$, FROM TABLE 5. THICKNESS INCLUDES 1 IN. FROM CENTER OF STEEL TO BOTTOM OF SLAB.

Span, in feet.	Total Uniform Load, in Pounds per Square Foot. 120		150		175		200		225		250		300		350	
	T, Thickness of slab, in inches.	W, Quantity of steel, in pounds per square foot of area.	T.	W.	T.	W.	T.	W.	T.	W.	T.	W.	T.	W.	T.	W.
5.0	3.3	0.32	3.6	0.36	3.8	0.39	4.0	0.41	4.2	0.44	4.3	0.46	4.6	0.50	4.9	0.54
5.5	3.5	0.35	3.8	0.39	4.0	0.41	4.3	0.46	4.5	0.48	4.6	0.50	5.0	0.55	5.3	0.59
6.0	3.8	0.39	4.1	0.43	4.3	0.45	4.6	0.50	4.8	0.52	5.0	0.55	5.4	0.61	5.7	0.65
6.5	4.0	0.41	4.3	0.46	4.6	0.50	4.9	0.54	5.1	0.57	5.3	0.59	5.7	0.65	6.1	0.70
7.0	4.2	0.44	4.6	0.50	4.9	0.54	5.2	0.58	5.4	0.61	5.6	0.64	6.1	0.70	6.5	0.76
7.5	4.4	0.47	4.9	0.54	5.2	0.58	5.5	0.62	5.7	0.65	6.0	0.69	6.5	0.76	6.9	0.81
8.0	4.7	0.51	5.1	0.57	5.4	0.61	5.8	0.66	6.0	0.69	6.3	0.73	6.8	0.80	7.3	0.87

$d = 0.042\,l\,\sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.138\,d$.

TABLE 10.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{250} = 64$; $\frac{E_s}{E_c} = 30$; $y_1 = 0.32\,d$; $y_2 = 0.68\,d$, FROM TABLE 5. THICKNESS INCLUDES 1 IN. FROM CENTER OF STEEL TO BOTTOM OF SLAB.

Span, in feet.	Total Uniform Load, in Pounds per Square Foot. 120		150		175		200		225		250		300	
	T, Total thickness of slab, in inches.	W, Quantity of steel, in pounds per square foot of area.	T.	W.	T.	W.	T.	W.	T.	W.	T.	W.	T.	W.
5.0	3.6	0.28	4.0	0.31	4.2	0.33	4.4	0.35	4.6	0.37	4.8	0.39	5.2	0.43
5.5	3.9	0.30	4.3	0.34	4.5	0.36	4.8	0.39	5.0	0.41	5.2	0.43	5.6	0.47
6.0	4.2	0.33	4.5	0.36	4.8	0.39	5.1	0.42	5.3	0.44	5.6	0.47	6.0	0.51
6.5	4.4	0.35	4.8	0.39	5.2	0.43	5.4	0.45	5.7	0.48	6.0	0.51	6.4	0.55
7.0	4.7	0.38	5.1	0.42	5.5	0.46	5.8	0.49	6.1	0.52	6.3	0.54	6.9	0.60
7.5	5.0	0.41	5.4	0.45	5.8	0.49	6.1	0.52	6.4	0.55	6.7	0.58	7.3	0.64
8.0	5.2	0.43	5.7	0.48	6.1	0.52	6.5	0.56	6.8	0.59	7.1	0 62	7.7	0.68

$d = 0.0482\, l \sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.102\, d$.

TABLE 11.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{200} = 80$; $\frac{E_s}{E_c} = 30$; $y_1 = 0.27\,d$; $y_2 = 0.73\,d$, FROM TABLE 5. THICKNESS INCLUDES 1 IN. FROM CENTER OF STEEL TO BOTTOM OF SLAB.

Span, in feet.	Total Uniform Load, in Pounds per Square Foot. 120		150		175		200		225		250		300	
	T, Total thickness of slab, in inches.	W, Quantity of steel, in pounds per square foot of area.	T.	W.	T.	W.	T.	W.	T.	W.	T.	W.	T.	W.
5.0	4.2	0.22	4.6	0.25	4.8	0.26	5.1	0.28	5.4	0.30	5.6	0.32	6.0	0.36
5.5	4.5	0.24	4.9	0.27	5.2	0.29	5.5	0.31	5.8	0.33	6.0	0.34	6.5	0.38
6.0	4.8	0.26	5.3	0.30	5.6	0.32	5.9	0.34	6.2	0.36	6.5	0.38	7.0	0.41
6.5	5.1	0.28	5.6	0.32	6.0	0.34	6.3	0.36	6.7	0.39	7.0	0.41	7.5	0.45
7.0	5.4	0.30	6.0	0.34	6.4	0.37	6.7	0.39	7.1	0.42	7.4	0.44	8.0	0.48
7.5	5.8	0.33	6.3	0.36	6.8	0.40	7.2	0.43	7.5	0.45	7.9	0.47	8.5	0.52
8.0	6.1	0.35	6.7	0.39	7.1	0.42	7.6	0.45	8.0	0.48	8.3	0.50	9.0	0.55

$d = 0.058\, l \sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.068\, d$.

TABLE 12.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{500} = 32$; $\frac{E_s}{E_c} = 15$; $y_1 = 0.32\ d$; $y_2 = 0.68\ d$, FROM TABLE 5. THICKNESS INCLUDES 1 IN. FROM CENTER OF STEEL TO BOTTOM OF SLAB.

Span, in feet.	Total Uniform Load, in Pounds per Square Foot.															
	150		175		200		225		250		300		350		400	
	T, Total thickness of slab, in inches.	*W*, Quantity of steel, in pounds per square foot of area.	*T*.	*W*.	*T*.	*W*.	*T*.	*W*.	*T*.	*W*.	*T*.	*W*.	*T*.	*W*.	*T*.	*W*.
5.0	3.1	0.43	3.3	0.47	3.4	0.49	3.6	0.53	3.7	0.55	4.0	0.61	4.2	0.65	4.4	0.69
5.5	3.3	0.47	3.5	0.51	3.6	0.53	3.8	0.57	4.0	0.61	4.3	0.67	4.5	0.71	4.7	0.76
6.0	3.5	0.51	3.7	0.55	3.9	0.59	4.1	0.63	4.2	0.65	4.6	0.73	4.8	0.78	5.1	0.84
6.5	3.7	0.55	3.9	0.59	4.1	0.63	4.3	0.67	4.5	0.71	4.8	0.78	5.2	0.86	5.4	0.90
7.0	4.0	0.61	4.1	0.63	4.4	0.69	4.6	0.73	4.8	0.78	5.1	0.84	5.5	0.92	5.8	0.98
7.5	4.2	0.65	4.4	0.69	4.6	0.73	4.8	0.78	5.0	0.82	5.4	0.90	5.8	0.98	6.1	1.04
8.0	4.4	0.69	4.6	0.73	4.8	0.78	5.1	0.84	5.3	0.88	5.7	0.96	6.1	1.04	6.4	1.10
8.5	4.6	0.73	4.8	0.78	5.1	0.84	5.3	0.88	5.6	0.94	6.0	1.02	6.4	1.10	6.8	1.18
9.0	4.8	0.78	5.0	0.82	5.3	0.88	5.6	0.94	5.8	0.98	6.3	1.08	6.8	1.18	7.1	1.25
9.5	5.0	0.82	5.3	0.88	5.6	0.94	5.8	0.98	6.1	1.04	6.6	1.14	7.1	1.25	7.5	1.33
10.0	5.2	0.86	5.5	0.92	5.8	0.98	6.1	1.04	6.4	1.10	6.9	1.20	7.4	1.31	7.8	1.39

$d = 0.34\ l\ \sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.204\ d$.

curve where the modulus of elasticity of concrete is supposed to be constant. If calculations are made for values about the elastic limit, then, according to some investigations, the moduli are variable in the concrete from the extreme fiber to the neutral axis, and the stress diagram would be somewhat like that shown in Fig. 18. With the latter assumption, the resisting moment is $12 \times f_c \times \frac{2}{3} y_1 \left(\frac{5}{8} y_1 + y_2\right)$, instead of $12 \times f_c \times \frac{y_1}{2} \left(\frac{2}{3} y_1 + y_2\right)$.

Quantity of Steel Reinforcement.—The force from the concrete (12 in. wide) resisting the bending moment due to the loading is $12 \times f_c \times \frac{y_1}{2}$; this belongs to a couple, and must equal the resistance of $f_s \times a$ in the steel. Hence, $a = 6\ \frac{f_c}{f_s}\ y_1$, in square inches, or, in pounds per square foot,

$$a = 20.4\ \frac{f_c}{f_s}\ y_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Substituting the values of y_1 and $\frac{f_c}{f_s}$ found in connection with each table, the quantity of reinforcement for each thickness of slab is derived.

TABLE 13.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{650} = 24.6$; $\frac{E_s}{E_c} = 15$; $y_1 = 0.38\,d$; $y_2 = 0.62\,d$, FROM TABLE 5. THICKNESS INCLUDES 1 IN. FROM CENTER OF STEEL TO BOTTOM OF SLAB.

Span, in feet.	Total Uniform Load, in Pounds per Square Foot. 150		175		200		225		250		300		350		400	
	T, Total thickness of slab, in inches.	*W*, Quantity of steel, in pounds per square foot of area.	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*
5.0	2.7	0.54	2.9	0.60	3.0	0.63	3.1	0.66	3.2	0.69	3.4	0.76	3.6	0.82	3.8	0.88
5.5	2.9	0.60	3.0	0.63	3.2	0.69	3.3	0.72	3.4	0.76	3.7	0.85	3.9	0.91	4.1	0.98
6.0	3.1	0.66	3.2	0.69	3.4	0.76	3.5	0.79	3.7	0.85	3.9	0.91	4.1	0.98	4.4	1.07
6.5	3.2	0.69	3.4	0.76	3.6	0.82	3.7	0.85	3.9	0.91	4.2	1.01	4.4	1.07	4.6	1.13
7.0	3.4	0.76	3.6	0.82	3.8	0.88	3.9	0.91	4.1	0.98	4.4	1.07	4.7	1.17	4.9	1.23
7.5	3.6	0.82	3.8	0.88	4.0	0.94	4.1	0.98	4.3	1.04	4.6	1.13	4.9	1.23	5.2	1.32
8.0	3.7	0.85	4.0	0.95	4.2	1.01	4.3	1.04	4.5	1.10	4.9	1.23	5.2	1.32	5.5	1.42
8.5	3.9	0.91	4.1	1.00	4.4	1.07	4.6	1.13	4.8	1.20	5.1	1.29	5.4	1.39	5.8	1.51
9.0	4.1	0.98	4.3	1.04	4.6	1.13	4.8	1.20	5.0	1.26	5.4	1.39	5.7	1.48	6.0	1.57
9.5	4.3	1.04	4.5	1.10	4.8	1.20	5.0	1.26	5.2	1.32	5.6	1.45	6.0	1.57	6.3	1.67
10.0	4.4	1.07	4.7	1.17	4.9	1.23	5.2	1.32	5.4	1.39	5.8	1.51	6.2	1.64	6.6	1.76

$d = 0.028\,l\,\sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.315\,d$.

Excessive Concrete or Steel.—To note the effect of excessive concrete or of excessive reinforcement on the carrying capacity, observe y_1, Equation 1. Assuming the working stress in the steel to be constant, y_1, in Table 5, increases with an increase in extreme fiber stress and decreases with an increase in the modulus of the concrete. Some designers use cinder concrete with an excess of steel in order to keep the slab as thin as possible.

Example 1.—Suppose we are limited to a working extreme fiber stress of 250 lb. per sq. in., are designing for a total floor load of 150 lb. per sq. ft. on a 6 ft. 0-in. span (simply supported slabs), but wish to limit the thickness of the slab to 4 in. (3 in. effective depth).

In this case, the extreme fiber stress being fixed at 250 lb. per sq. in., and the ratio of moduli being assumed as constant at 30, the variable to be sought in determining y_1 is the stress in the steel. Suppose we use sufficient reinforcement to bring y_1 midway between the center of the steel and the top of the slab.

Referring to Equation 1, and substituting values for d (3 in.), m (30), and y_1 ($1\frac{1}{2}$), we have $\frac{3}{2} = \frac{3}{\frac{n}{30}+1}$; but $n = \frac{f_s}{f_c} = \frac{f_s}{250}$; hence, $f_s = 7\,500$.

Therefore, in order to bring the neutral axis midway in the effective depth of the slab and not exceed 250 lb. per sq. in. in extreme fiber stress, the reinforcement must not be strained beyond an amount given by a stress of 7 500 lb. per sq. in. The weight of reinforcement in this case will be (Equation 7): $a = 1.02$ lb. per sq. ft. Now, with steel stressed to 16 000 lb. per sq. in., as assumed in Table 7, the total thickness of the slab would have been 5.3 in., but the quantity of steel would then have been only 0.44 lb. per sq. ft. Hence, 0.58 lb. per sq. ft. is required to save 1.3 in. in thickness of cinder concrete. With reinforcing steel at $2\frac{1}{2}$ cents per lb., the 0.58 lb. of excess steel will cost 1.45 cents per sq. ft.; and the saving effected will be 1.3 in. of concrete.

Example 2.—Suppose we have a 6 ft. 0-in., simply supported slab, and are using a cinder concrete having a working capacity of 250 lb. per sq. in. of extreme fiber stress (Table 7); and, for some reason, we wish to use a certain reinforcement, a little light in area, to take the tension at 16 000 lb. per sq. in. We are not allowed to exceed the 16 000 lb., and, therefore, must thicken the slab to more than the normal thickness given in Table 7; that is, we must use an excess of concrete.

If the total floor load is 150 lb. per sq. ft., we find in Table 7 that the slab will be 5.3 in. thick, and the reinforcement will weigh 0.44 lb. per sq. ft. of floor.

Assume that the reinforcement we wish to use weighs only 0.36 lb. per sq. ft. We see by Equation 7 that a decrease in the section of the reinforcement may be effected by a decrease in the stress on the extreme fiber of the concrete. Therefore, looking in Table 8, which has a less extreme fiber stress, we find that, with a slab 6.2 in. thick, the reinforcement is 0.36 lb. per sq. ft. of floor.

If a less quantity of reinforcement had been desired, then calculations using lower extreme fiber stresses, similar to the calculations used for making these tables, would have been necessary.

Stresses in Approved Floor Slabs.—The location of the neutral axis for any particular thickness of slab is a function of the relative stresses in the materials, and of the moduli of elasticity. Referring to Fig. 7, assume that the ratio of moduli remains constant at 30, and that, $\frac{f_s}{f_c} = 64$, that is $\frac{16\ 000}{250}$ or $\frac{20\ 000}{312\frac{1}{2}}$, and that the slab tested is continuous; we find in Table 10 (the table using these assumptions) that the neutral axis is 0.32*d* from the top of the slab when the concrete and steel are proportioned for these stresses. For a 4-in. slab, therefore, the neutral axis would be 0.96 in. from the top of the slab. In Table 10, a normal 4-in. slab is good for a total floor load of 150 lb. per sq. ft. on a 5 ft. 0-in. span, with a reinforcement of 0.31 lb. per sq. ft.; but in this 4-in. slab, of 8 ft. 0-in. span (Fig. 7), as approved, No. 5 wires at 3-in. centers are used. This makes the actual reinforcement about 50% greater than the normal reinforcement for Table 10.

This relative excess of steel (reducing the comparative strain coming on the reinforcement) will lower the neutral axis below that given in Table 10. The strain on the steel, $\frac{f_s}{E_s}$, therefore, is in this case, $\frac{2}{3} \times \frac{16\ 000}{30\ 000\ 000}$ (Fig. 16). Then, the neutral axis, on account of this excess of steel, will be located 0.41*d* or 1.23 in. from the top of the slab—instead of 0.96 in. where there is no excess of steel.

According to the ruling of the New York City Building Department, the 4 in. of cinder concrete weighs 32 lb. per sq. ft. of area. Therefore, the approval of this floor construction is for a total floor load of $250 + 32 = 282$ lb. per sq. ft.

Therefore, the bending moment, from this total load (if the slab is called continuous) is $\frac{W L^2}{1}$, or 18 048 in-lb.

This is equal to the resisting moment (one force of the couple being $12\ f_c\ \frac{1.23}{2}\Big)$ having a lever arm, $\frac{2}{3} \times 1.23 + 1.77$ in., or $19.11 \times f_c$, in inch-pounds. Therefore, $f_c = 944$ lb. per sq. in., which

is the extreme fiber stress on the concrete. The area of the reinforcement being 0.134 sq. in., $f_s = 52\,000$ lb. per sq. in. in the steel.

If, instead of considering the slab continuous, we take an end slab,* as is done with stone concrete, we have a moment of $\frac{12\ W\ L^2}{10}$, or 21 658 in-lb. In this case the extreme fiber stress will be 1 133 lb. per sq. in., and the stress on the steel, 62 000 lb. per sq. in.

If we take the condition of reinforcement simply laid over the supports, as shown in Figs. 14 and 15, then the slab is simply supported at the ends, and the load moment becomes $\frac{12\ W\ L^2}{8}$, or 27 072 in-lb. This latter condition gives an extreme fiber stress of 1 417 lb. per sq. in., and a steel stress of 78 000 lb. per sq. in.

Of course, under such abnormal stresses the modulus of elasticity for cinder concrete, probably, does not remain constant, and the resistance diagram may assume a shape similar to Fig. 18. This would slightly modify the foregoing deductions. However, even if an indefinite number of assumptions were made regarding the ratio of moduli of elasticities of wire and of cinder concrete, ratios of stresses in concrete and steel, and the variable support of slabs, etc., the foregoing deduced stresses would still be found to represent approximately correct conditions.

Without further attempt to lengthen this inquiry (by considering an indefinite number of cases and conditions), the writer would call attention to the comparison of the thickness of stone concrete slabs with approved cinder concrete slabs under similar conditions. It is found that, for the same conditions as the approved cinder concrete slab (Fig. 7) discussed herein, the stone concrete would be required to be about 1 in. thicker than the cinder concrete.

Recommendations.—

1.—The practice of testing cinder concrete, arbitrarily, under the most favorable conditions, and then recommending the results of these tests to be used under the most unfavorable conditions which may exist in a building, is dangerous and entirely unnecessary.

2.—Unit stresses for the design of reinforced cinder concrete are of more importance for its proper use than the unit stresses

* Progress Report, Special Committee on Concrete and Reinforced Concrete, *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 385.

used for reinforced stone concrete, because of the greater variableness of the former.

3.—There are, at present, sufficient precedents for choosing some unit stresses for the use of reinforced cinder concrete slabs in each locality where this material is being used.

4.—It is practical, and of little cost for each city, to establish ultimately proper working stresses for reinforced cinder concrete, based on tests with the materials used in their locality.

5.—With cinder concrete used according to proper working stresses, the construction will be removed from the realm of mystification and of misrepresentation to the field of Civil Engineering.

DISCUSSION

Mr. Diamant.

ARTHUR H. DIAMANT, ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read this paper with great interest, and, having had experience during the past 5 years in the use of reinforced cinder concrete, agrees with the author that the present method of tests of floor slabs does not conform to conditions on construction work.

Aside from the design, however, the great fault in the construction of such floors is the lax supervision in placing the material. The majority of apartment and loft buildings in New York City are erected by speculators. The men in charge of the erection of these structures for the builders know very little about reinforced concrete work, and are rarely on the floors where the arches are being placed. The inspector of the Building Department has so much work to supervise, that he can devote very little time to this part of his duties, and the result is that there is too much dependence on the contractor to do good work. His men, usually unskilled laborers, are required to fill a certain number of centers in a specified time, and, in order to do this, sufficient care is not taken to place the reinforcing metal properly. With the exception of wire-cloth, the spacing of the metal is usually guessed at, and thus its weight per square foot of arch is often insufficient.

Another cause of trouble is in the mixture of the cinder concrete. When hand mixing is done, there is very little attempt to measure the ingredients, and, in machine mixing, the cement, sand, and cinder hoppers are not kept filled. The writer was on the tenth floor of a hotel, being erected under his supervision, when he noticed that the cinder concrete which was being hoisted did not seem to have the proper proportion of cement. The contractor had an efficient mixer plant in the cellar, and when the cause of the poor mix was investigated, it was found that the laborers were so busy keeping the sand and cinder bins filled that they neglected the cement bin.

One of the most pernicious faults is the cutting of holes in concrete arches shortly after they are laid. In an inspection of a large loft building, the writer condemned a panel of reinforced cinder concrete floor which adjoined a stair-well. A large hole had been cut through this panel for a 5-in. soil pipe, and smaller holes for the passage of water, steam, and electric pipes. The reinforcing rods had been cut and bent so that they would not interfere with the pipes. Before the arch could be taken down, it failed because a barrel of lime was rolled over it. There is no excuse for work of that kind, for, in a properly designed structure, the location of all pipe lines is known, and openings can be left in the arches when they are poured.

In conclusion, the writer wishes to state once more that, even though the floor slab is well designed, the careful work of the engineer is not properly carried out because of inefficient supervision in the field. Mr. Diamant.

J. R. WORCESTER, M. AM. SOC. C. E. (by letter).—The argument of the author in favor of establishing allowable unit stresses and physical properties of cinder concrete as a means of calculating safe loads for certain designs is sound and convincing. Though there are some difficulties in the way of reaching satisfactory units, they should be overcome if we consider that the margin of inaccuracy will be much less than in any attempt to determine strength by crude, isolated, field tests of actual construction. Mr. Worcester.

The chief uncertainty with regard to the material is on account of the wide range in quality of cinders, running, as they do, all the way from a soft, impalpable ash to a vitrified clinker. The writer has not seen a satisfactory specification for cinders, by which good material for concrete can be distinguished from bad. It is easy enough to write one which will exclude bad material, but the trouble with it would be that it would be so stringent that no material could be obtained which would comply with it. It is to be hoped that further light will be shed on this subject. Another uncertainty in the value of the best cinders lies in the great irregularity in the proportion of fine particles, corresponding to sand, which is contained in every load. This variation is so great in practice that frequently a batch of concrete may contain scarcely any material which could be classed as coarse aggregate. Under such circumstances we would obtain for a product, using 2 parts of sand and 5 of cinders, a 1:7 mortar, of which part of the aggregate is less hard than sand. It is chiefly to these peculiarities that we may attribute the wide discrepancies in the results of experiments with cinder concrete, as shown in Table 3, and the general disrepute which the material has among structural engineers.

It is true that cinder concrete slabs, in spite of their disadvantages, have been used to a phenomenal extent, and it is also a fact that failures which can be shown to be due to the use of this material are remarkably few in number. This being the case, it certainly behooves the Profession to deal with the subject in a tolerant spirit, and to do what it can to lay down rules for its safe use.

This Society's Special Committee on Concrete and Reinforced Concrete, in its report presented in 1913 prescribed units, which, though tentative, seem to correspond fairly well with present practice. The subject of the modulus of elasticity of cinder concrete is not referred to in that report, presumably being classed with concretes having an ultimate strength of less than 2 200 lb. per sq. in., for which the ratio of moduli is to be taken as 15. This low value of the ratio does not correspond with general practice. With this exception, the

Mr. Worcester. rules laid down in the report cover the method of computation and manipulation quite comprehensively.

Although cinder concrete may prove entirely satisfactory in floor and roof slabs, it is altogether too uncertain a material to recommend for beams, girders, and other structural members, and it is objectionable even for slabs when the span is too great. The reason for this seems to be that the softness of the aggregate allows an inelastic set, which becomes apparent in the deflection when the span is sufficient. It may not be possible to make a hard and fast rule limiting the length of span, for in some places the deflection may not be of importance, or, it might be compensated by cambering the slabs, but the peculiarity should not be lost sight of.

Mr. Buel. A. W. Buel, M. Am. Soc. C. E.—When safe working stresses for cinder concrete have been correctly determined for various specified conditions encountered in practice, it will without much doubt become clearly apparent that there is little, if any, economy in its use for structural elements subject to stress, notwithstanding its lower weight per unit of volume as compared with stone concrete.

The speaker regrets that the author has not taken up the question of the corrosion of embedded steel, as it is of vital importance in a permanent structure, especially as the records of some experiments and considerable experience have shown bad results.

Although a satisfactory series of experiments from which to deduce safe unit stresses for cinder concrete would be a valuable addition to our knowledge of the subject, it would seem advisable first to determine by experiment and experience whether the material is suitable for permanent structures, and that the reinforcing metal is reasonably safe from corrosion. Until this is done, it will hardly be safe to specify cinder concrete for reinforced members or to adopt it as an acceptable material of engineering or of construction—except as a filler—no matter how satisfactory the experimental determinations of its strength may be. In other words, it seems to the speaker that the first experimental and research work with reinforced cinder concrete should be confined to the question of the corrosion of the reinforcing metal, and unless the results are satisfactory on this point it would be a waste of time and money to enter on elaborate experiments to determine proper unit stresses.

It has been suggested that corrosion sometimes takes place with steel embedded in stone concrete. This is quite true, but as far as the speaker has been able to ascertain corrosion of reinforcing metal in good stone concrete is almost invariably due to one of two causes: electrolysis or palpable defects in design or construction, for both of which there are practical protective measures affording reasonable security. Certainly, one can hardly hope for absolute security, but

there is evidence that iron embedded in stone concrete has been preserved rust free for periods as great as 400 years. Mr. Buel.

The following notes relate to the corrosion of reinforcing metal embedded in cinder concrete: Mr. William H. Fox* states that the results of experience and experiment show serious corrosion in almost all cases, and recommends that with cinder concrete no mixture less rich than 1:1:3, or one still richer in cement, should be used. In a report of a committee read before the Structural Association of San Francisco, it was stated that metal encased in cinder concrete had been corroded, and the committee advised a revision of the code, to exclude the use of cinder concrete. Mr. Fox reports a set of experiments at the Thayer School, with the following conclusion:

"With but one exception, one or more of the three steel pieces in each specimen showed unmistakable signs of corrosion. Apparently it made no difference how the concrete was mixed—wet or dry, tamped or untamped; whether the steam or water treatment was used, the result was the same—rust streaks and spots were found, the difference in the amount of corrosion being imperceptible."

These records seem to make a case against cinders for reinforced concrete that will require something more than explanation, and a good deal of substantial evidence to the contrary, before conservative engineers and architects can safely adopt this material. The advocates of cinder concrete should accept the task of making the necessary demonstrations.

Charles C. Hurlbut, Assoc. M. Am. Soc. C. E.—This paper takes a long step toward putting cinder concrete on a rational basis, like other materials of engineering. That it is a perfectly reliable material for many purposes, when made with a good grade of cinders, the speaker is convinced from his own experience. It is not suitable for any work requiring density, such as foundations or outer walls, neither is it suitable for reinforced beams and girders where a stronger material is required; but for floor-slabs between steel beams, as in modern skeleton construction, with relatively short spans and moderate loads, it is very satisfactory. It is not because a great deal of cheap and shoddy work has been done with it, but in spite of that fact, that 90% of the fire-proof floor construction in New York is of cinder concrete. The speaker has repeatedly seen new cinder concrete floors in buildings under construction carry loads enormously in excess of those for which they were designed or would ever be called on to carry in actual use. Any one having much experience with cinder concrete in floor-slabs is sure to acquire considerable respect for its load-bearing qualities. Of course, a slab of stone concrete of the same thickness is much stronger, but if the slab is stronger than necessary, there is obviously no gain in increasing the strength, Mr. Hurlbut.

* *Engineering News*, May 23d, 1907.

Mr. Hurlbut. and usually it would not be advisable to decrease the thickness much, even if stone were used.

On the other hand, there is no doubt that there has been a large quantity of very inferior cinder concrete work. Sometimes this is the result of direct "skinning" of the mix, and sometimes it is due to the use of unsuitable cinders. Good cinders are sometimes hard to obtain, even with the best intentions, and on many occasions the speaker has rejected cinders on the job which otherwise would have been used in the floor construction. It may be noted in passing that the floor construction in question had been duly "passed" by the Building Department, but it is safe to say that the test on which the approval was based was made with very different cinders from those rejected, illustrating Mr. Waite's objection that the Building Department tests have small relation to what may be expected of any particular floor. The cinders rejected were, in some cases, full of unburned coal and, in other cases, little but fine ashes. Wherever cinders are used they should be subject to rigid inspection. In the matter of unburned coal in cinders, although it is the speaker's practice not to accept cinders containing more than the small necessary proportion, it is his opinion that the danger from unburned coal in concrete is over-estimated. This opinion is based on a rather crude test made on a partition built of 2-in. cinder concrete blocks. The blocks were literally filled with unburned coal. A small covered enclosure was built and subjected for more than an hour to a very hot wood fire. The coal on the surface was reduced to cinder, but all below the surface seemed to be entirely unaffected.

The objection has been raised, in designing cinder concrete by formula instead of by test, that the material is too variable. This argument does not seem to be well founded. Given a certain system of reinforcement, the strength of any floor-slab will vary from that developed by a full-sized test sample in just the same degree that the materials in it vary from those in the slab tested. It is just as proper, therefore, to designate a safe unit compressive strength and other properties for cinder concrete as it is to assume a safe slab strength based on a test, and far more rational. As a matter of fact, it is a far safer method, for the average strength of cinder concrete can be determined with some approximation to the truth by taking a large number of tests on varying grades of cinders. On the other hand, a test made on a slab proves little beyond the slab tested. Only one test is made on each "system", and it is safe to say that the cinders entering into these tests are the best that can be obtained. The speaker has witnessed a number of these tests; he has seen some of the test slabs built, and knows that the cinders were better than the average used throughout the city. The charge is not made that any of the tests have been "faked", or even that the materials used were not

such as can be obtained in the open market, but merely that the tests of which the speaker has knowledge were made with rather exceptionally clean cinders, which are not always used in actual construction. No proof as to the accuracy of an engineering formula would be deemed conclusive if based on a single test, and yet a single test with an admittedly variable material is here used as a pattern to be followed without variation. It would be possible to define a standard cinder aggregate, the properties of which would not vary beyond reasonable limits, no more, for instance, than timber varies, or some other materials the use of which is based on rational formulas. The proportion of fine ash can readily be determined with a sieve of some standard mesh, and the percentage of unburned material can be determined with sufficient accuracy by means available almost anywhere. These are the two principal uncertain and variable elements in cinder as used in concrete. Mr. Hurlbut.

A serious objection that has been raised to the use of cinders is their alleged tendency to corrode metal with which they come in contact, and it is to be regretted that the author did not take up this phase of the question. It has been the speaker's opinion that a cinder concrete, if mixed not leaner than 1: 2½: 5 and placed rather wet, would not injure pipes or steel reinforcement embedded in it. No cases of corrosion clearly attributable to the cinders have come to his attention under these conditions. There are many cases, however, where pipes, embedded in weak cinder fill, have been destroyed, and the presumption has been that the cinders were to blame. It is the speaker's practice to specify and enforce rigidly the requirement that all pipes embedded in the floor construction or fill, where cinders are involved, are to be solidly encased in cement mortar made with 1 part of cement to 2 parts of sand. The Lackawanna Railroad has experienced considerable difficulty with the corrosion of pipes in its stations which, in some cases at least, the engineers have attributed to the effects of contact with cinder concrete. Owing to this belief, they are now prohibiting cinder concrete in every form. In this case, at least some of the trouble was subsequently found to be the result of electrolysis. How much of the corrosion, if any, was actually due to cinders, it would be hard to say. It is quite possible that in other cases of damage to pipes, cinder concrete has been made to bear the blame which should have been attributed to stray currents of electricity.

W. B. Claflin, M. Am. Soc. C. E.—The use of reinforced cinder concrete for floor-slabs and as a fire resistant has reached such proportions, especially in Eastern cities, that its importance as a structural material should be brought to the attention of the Profession at large, and Mr. Waite's paper will be instrumental in accomplishing this. Mr. Claflin.

Some time prior to the introduction of cinder concrete in New York City, architects had become more or less dissatisfied with the

Mr. Claflin. materials and methods of fire-proofing then in common use. They were either cumbersome and heavy, or proved inefficient when subjected to great heat. The economic conditions which caused the development of tall buildings and the necessity for reducing the dead loads for the purpose of eliminating the excess of steel in the frames, made it desirable to obtain a material that was durable, fire-resisting, and light. About 16 or 17 years ago, cinder concrete, having been officially tested and approved by the New York Building Department, was introduced. That it has been a success in its limited field of application, time has shown conclusively. The method of testing originally devised, and still followed, does not give accurate results, but was the best that knowledge then available afforded.

For several years after its introduction, the cinder concrete industry was dominated by reputable individuals and companies of financial responsibility who generally controlled a patented method of reinforcing which was not available to others in the trade. It was their custom, as a rule, to arrange for a steady supply of cinders from fairly reliable sources.

The patents referred to cover the forms of reinforcing bars and rods, either plain or deformed, with spacers, clips, and hangers for insuring the proper placing of the reinforcement and holding the concrete protecting the bottom flanges of the steel floor-beams and girders. There is also a rectangular wire mesh with welded intersections, which is one of the best types of reinforcement.

Each concern conducted a series of tests under the direction of the Building Department, and obtained approvals on forms of construction which gave the architects a range of choice sufficiently wide to cover generally all the requirements of practice. The conditions in the trade were fairly stable, with reasonable competition, a good grade of cinders, and responsible concerns to deal with.

Five or six years ago a subsidiary of the Steel Corporation began the manufacture of a triangular-mesh, wire reinforcement, which, having been tested and approved in slabs of various spans and for various quantities of steel, was placed on the market. This is sold to any one having the means to pay for it, and is so cheap that, in many instances, as a result of the ensuing sharp competition, the proprietors are obliged to use it instead of their own methods of reinforcement. No criticism of this form of reinforcement is here implied, for it is one of the best in use to-day.

The trade is not now in such a satisfactory condition as formerly, for the reason that an irresponsible, unskilled, and, too frequently, careless element has appeared, which, through ignorance or willingness to assume risks, underbids its more conservative and responsible competitors. It is practically impossible for the architect to induce the average owner (especially the speculator) to discriminate between the

two elements in the trade. Cinder concrete is cinder concrete to him, and it is difficult for him to see beyond the dollars apparently saved by the acceptance of an unduly low estimate. Mr. Claflin.

In times of normal building activity, it is difficult and sometimes impossible for the new element in the trade to obtain cinders of good quality; consequently, they are often taken from household refuse at the dumps, and contain all sorts of foreign matter adversely affecting their quality and the strength of the concrete.

The foregoing has been written for the purpose of showing clearly the influences which have resulted in conditions as they are to-day.

For some time the method of arbitrary tests imposed by the Bureau of Buildings has caused dissatisfaction, not only in the trade, but among architects, because they require much time and are expensive, and because the approval is limited to the exact form tested, no variation whatever being allowed. This means that a firm desiring to use a number of forms of construction must take the time for, and incur the expense of, a test on each one, no matter how slight the difference may be.

This condition has naturally led to the desire, on the part of the cinder concrete interests, for a more rational and simple method of determining the strength of slabs, and the proposal to calculate their stability on the basis of unit stresses is the result. This proposal is logical and highly desirable, if it can be done with safety, but there are points involved which would seem to make it unwise.

Until recently all slabs were tested under uniformly distributed loads, developing unbelievable carrying capacities. The only way to account for this is that the load arches itself, a large and indeterminate portion being transferred to the haunches. Reference to Fig. 2 will show at a glance that arching occurs. In fact, it would require great skill in placing the load to prevent it. We are in the dark, therefore, as to the true strength of the slabs we have used. Within the last two or three years slabs have been tested under concentrated loads at the middle.

As has been intimated, it is difficult to obtain cinders of anywhere near a standard quality. In one building, where the cinder concrete would not set, investigation showed that, although apparently clean, on being wet, strong fumes of ammonia gas were liberated. Further inquiry showed that the cinders had come from a brewery in which one of the ammonia pipes had burst over the place where they were stored. In another case, it was found that the cinders (in some concrete which would not set) had come from a sugar refinery and were impregnated with molasses. Fortunately, the work was being done by reputable concerns, and the faulty cinders and concrete were removed, care being taken to have them clean thereafter.

Mr. Claflin. Portland cement is now a standard material, and can hardly be considered an element of uncertainty except in cases where it is not properly protected prior to use, or is mixed in improper proportions. This may be generally applied to the sand used in the city.

The proper inspection of cinder concrete requires a superintendent continually on the work to see that the materials are of the proper kind, and that they are mixed and placed as they should be. It is practically impossible to persuade an owner to incur the expense of a clerk of the works, especially in commercial work. This is another valid reason for a strict law safeguarding the use of cinders.

The most uncertain of all the elements entering into the composition of any concrete, but more especially cinder concrete, is the so-called human element. Carelessness in or indifference to any one of the many precautions which must be observed in the mixing and placing of concrete may lead to failure.

If unit stresses are used in the calculation of cinder concrete slabs, they should be low enough to discount faulty cinders, carelessness, possible dishonesty, indifference, and lack of inspection. To accomplish this, the unit stresses must be so low that the cinder concrete interests themselves would object on the ground that the quantity of material required would be increased to such an extent that they could not compete successfully with other forms of floor construction. If unit stresses are used, the cinders should be standardized; the law should require that they be washed and screened.

The fire-resisting qualities of cinder concrete have been developed so many times in tests and during actual fires that it would seem that the time has come when they may be accepted as proved and fire tests eliminated in the future.

Recommendations.—The speaker makes the following recommendations:

1. That the Bureau of Buildings conduct a series of tests, approximating as closely as possible commercial conditions in the mixing and placing.

2. That the slabs be tested under concentrated loads at the middle of the spans, with various combinations of span lengths, thicknesses of slab, and areas of reinforcement.

3. That the results of these tests be divided by a conservative factor of safety and the remainders be tabulated for use as the basis of slab design. For instance, if it is desired to use a slab 4 in. thick, with a 6-ft. span, carrying an applied load of 150 lb. per sq. ft., a given area of reinforcing steel per foot of width should be required.

4. That a ruling be made requiring the washing and screening of all cinders used in all future buildings.

5. That all existing approvals be revoked, after the results of the new tests are available.

The adoption of these recommendations would place the use of cinder concrete on a rational basis and enable those interested to proportion the slabs with a degree of freedom and safety hitherto unknown. Mr. Claflin

The corrosion of steel embedded in cinder concrete has been mentioned. Several years ago A. N. Talbot, M. Am. Soc. C. E., embedded steel in cinder concrete and concluded that the corrosion was negligible, because, with the wet mixtures used, a film of cement was deposited around the steel which protected it against the action of any chemicals in the cinders tending to cause corrosion.

It is not safe to place unprotected pipes in the cinder fill over slabs. This can probably be explained by the fact that the mixture is usually 1:7 or more, and is placed quite dry, the consequence being that there is not sufficient water present to carry in suspension enough cement to coat the pipes. Piping thus placed should always be given a heavy coat of asphaltic or similar acid-resisting paint.

About the only knowledge we have of the effect of cinder concrete on reinforcing steel in actual construction is derived from the Pabst Building, which was removed (to make way for the Times Building) after having been occupied for 7 years. Corrosion of the steel was found in a few places, but not to a degree affecting its strength. Pipes embedded in the cinder fill were badly corroded.

F. W. Skinner, M. Am. Soc. C. E.—The speaker's recollection is that, when cinder concrete was first used, it was very largely promoted and intended chiefly as a filler, a light cushion, or a cover for other construction, and it was not expected that it should be subjected to very heavy stresses or that great responsibility should be placed on it. It was found to be convenient, light, and cheap, but it has had larger and larger duties placed on it ever since. Mr. Skinner.

As regards construction, however, concrete of the best quality is one of the most important engineering materials, and one which probably more than any other, varies and is subject to very great modifications through its components and through the manner of its manipulations; it cannot be made and inspected too carefully. Stone concrete without reinforcement is very different from cinder concrete, which is lighter and of more uncertain proportions, and is, and should be, chiefly adopted for mass and fill and for reduction of weight.

It seems to the speaker that any discussion which gives cinder concrete equal technical weight with a carefully prepared stone concrete, which can be subjected to thorough tests and be made a permanent material of construction, is hardly fair. It is carrying the question beyond the limit of technical considerations, even though it is not beyond the manner in which it is used. In this discussion the most important consideration in regard to cinder concrete is whether it is used to-day in the proper manner and for the proper purposes.

Mr. Lowinson.

OSCAR LOWINSON, ASSOC. M. AM. SOC. C. E.—As an authority on building codes and also on the fire-proofing of buildings, Mr. Waite has no superior. His experience in concrete construction and in the formulation of requirements in building-code matters, is such that his argument—that definite standards for reinforced cinder concrete construction should be established on a mathematical or scientific basis, rather than as a result of tests—entitles him to a respectful hearing.

One great problem in New York City, however, has been to try to make laws which will meet the conditions in the trade; and though this discussion as to the strength of a cinder concrete arch is all right when one knows what material enters into the structure, the trouble has been that frequently the material used is not cinder concrete as an engineer understands the term.

This unfortunate condition of affairs has arisen from the fact that the majority of the concerns who are engaged in cinder concrete construction are utterly irresponsible, have no engineers in their employ, and are subject to no supervision other than that casually given by the building inspector, who usually has only a few moments in the day to devote to the building. Consequently, those familiar with the conditions marvel at the few accidents that occur in New York City, for it is well known that, if it were possible to determine thoroughly the strains in the material which is called concrete, in a large number of cases, it would be found to be strained far beyond the breaking value due solely to the utter disregard of the requirements for reinforced cinder concrete construction.

The expedient was adopted of having tests and giving permits for each type of construction, and for a while this method was effective in that it compelled the installation to correspond to an approved test. Later, however, it was found that the concerns who complied with the tests could no longer compete in construction with the contractors who were willing to install, and the work has gradually drifted so that the concerns who make the tests, to-day, only sell the reinforcement, and the construction is done by other parties.

Under such conditions it is impossible to adopt any kind of a standard, either so-called scientific or practical, of any value; and it is doubtful whether it would be an improvement to change the method of requiring a standard. This certainly is a hardship to a conscientious contractor. It is only one of the unfortunate things in the building trade.

This subject is not new, as discussion on the formulation of data with reference to reinforcement has been going on since 1907. New York City has had five proposed building codes, and another is now under way. It is hoped that a building code will be established which will eliminate standards of this type entirely, leaving their determination to the formulation of regulations by the authorities. Possibly,

by such a method, a building code may be adopted which will be based, more or less, on rational ideas. Mr. Lowinson.

Heretofore, the difficulty has been the utter irresponsibility of those who are putting in the material and the lack of inspection of the material going into the structure. As a result of this experience it was determined years ago, as Mr. Waite has stated, that it was better to depend on tests with a material of this sort, and establish standards from the basis of such tests, than to attempt to establish standards or unit stresses in a combination material, like reinforced cinder concrete, and the authorities have been very slow in showing any sympathy with the idea that they can establish standards with reference to this mixture.

With a stone mixture one can establish a standard and can generally feel satisfied that the results will be fairly in accord with it. The aggregate is dense, and there is no possibility that it will disintegrate if the slightest inspection is undertaken. With cinders, however, one does not know what to expect; and it is on that account, more than anything else, that the standards in the code of 1912 were established after considerable discussion as to whether or not unit stresses should be permitted in the reinforcement of cinder concrete, and it would have been possible under a similar code, revised as a result of the discussion that took place after the publication of the original code, to establish certain standards of material made on a basis more or less scientific.

The speaker does not think it advisable to establish standards of that sort in any form, other than with reference to the thickness of the material and the quantity of reinforcement per unit or per square foot, by establishing a minimum, because of the fact that with such materials and with mechanics of the type engaged in that work in New York City, to-day, it is not advisable to go further.

With reference to the corrosion of pipes when buried in concrete, experience has taught us that the only way to feel sure that pipes will last is to insulate them, and thereby prevent the corrosion from getting at them. A wrought-iron steam pipe, embedded in a concrete floor, is good for from 2 to 5 years. It is the custom, especially in cellars, to try to keep the return pipe above the floor level. Corrosion in pipes is practically due to oxidation, caused by the sulphur in the cinders, or by the porosity of the concrete permitting the moisture, which is alkaline, to attack the pipe.

There has been a great deal of trouble with hot-water supply pipes, when buried; they corrode more rapidly than the cold-water pipes. Frequently, over these pipes there is a layer of plaster of Paris, or cement from a tile floor, and the corrosion is generally attributed to the above-mentioned cause, to which is added expansion and contraction of the pipe caused by variations in temperature.

Mr. Lowinson. It has been found that corrosion occurs on the outside and also on the inside of the pipe. Chemists may be able to explain why the corrosion takes place on the inside as well as on the outside, and why it is more active with hot-water than with cold-water pipes.

Mr. Strehan. GEORGE E. STREHAN, JUN. AM. SOC. C. E. (by letter).—The author's object in recommending that the application of cinder concrete in fire-proof floor construction be placed on a so-called scientific basis is to be commended. However, the writer fails to find any plausible basis in the paper on which to establish constants for calculation and unit stresses for design. The author states that there are sufficient precedents for establishing such factors, but a careful perusal of the paper does not elicit much information of great value along this line. Table 3, showing the physical and elastic characteristics of cinder concrete, which the author presents, was taken from a preliminary report of an investigation being made at Columbia University, and these data in themselves are not sufficiently comprehensive to be used as a basis for design.

Cinder concrete, as applied in New York City and vicinity in fire-proof floor filling between steel beams, is a material with which the author should be thoroughly familiar from long years of experience, both as a contractor and as an "expert tester." The knowledge thus gained should have convinced him that such a floor, built between rigid I-beam supports on ordinary spans of from 6 to 8 ft., is an entirely different structure from reinforced concrete of the slab, beam, and girder type. On spans up to 8 ft., and with a thickness of not less than one-twentieth of the slab length, the writer's investigations have shown that such factors as arch action, tension in the concrete, and the trussing effect of continuous reinforcement, all of which the author neglects, are important features in floor construction of this type.

The author ridicules the term "continuity" thus used without apparently realizing that it is not the continuity of the concrete slab which is referred to, but the anchoring of the reinforcement over the steel beam, and although he has little faith in it, he still feels that such continuity obtaining in the test construction and not in actual buildings has resulted in the excessive load approvals. Based on both theory and test, it can be shown that this strength, of the worst condition that occurs in building construction, that is, the end slab adjoining an open bay or exterior wall, is about three-fifths of that of the test construction as ordinarily installed. The author failed to note that the tests conducted by the Bureau of Buildings, with the resulting approvals to which he refers, were made on the basis of a factor of safety of 10. This, in the end-panel condition of actual practice, would correspond to a factor of safety of 6 for the approved loads.

Mr. Strehan.

The writer does not undertake to defend the method of handling cinder concrete floor construction which obtains in New York City, but believes that a rational basis for design should be established. With this end in view, a paper is now in course of preparation which it is hoped will soon be ready for publication. The investigation has included a study of the preparation of the raw materials, with the object of improving the mixture. Very little, if any, benefit has been derived by screening the cinders; in fact, the resulting concrete has shown a decrease in strength when compared with the ordinary 1 : 2 : 5 mixture in which hard coal cinders, as received from the boiler, are used.

The author states that the form of reinforcement is immaterial. Exception must be taken to this statement, because, in some tests of slabs reinforced with plain and deformed rods, the plain squares and rounds failed by the slipping of the reinforcement. The question of adhesion must undoubtedly be taken into account in the design. The writer wishes to emphasize further the fact that if the reinforcement is anchored to the steel girders, the resulting construction differs radically from the type of slab, simply reinforced, on which the author bases his plea for design. The extension of cinder concrete to general application in slabs, beams, and girders should in no case be recommended, in view of the variableness of the material. Its use between steel beams as a special application has undoubtedly proved satisfactory; but, even in this case, there should be greater caution than is now generally shown in the selection and preparation of the raw materials for the concrete.

Mr. Himmelwright.

A. L. A. Himmelwright, M. Am. Soc. C. E. (by letter).—The subject of cinder concrete floors is one in which the writer is greatly interested. The idea of rationalizing the theory and practice of their construction is not new, and would be most desirable if a practical and correct formula could be evolved that would be generally applicable to the different forms of reinforcement and methods of using it. The writer spent much time years ago in attempting to develop such a formula, and was obliged to give up the attempt.

Mr. Waite's arguments are relevant, but, unfortunately, they are applicable only to the ordinary and usual types and methods of reinforcement, and would not include numerous others equally desirable.

Take, for example, an abnormal case, such as a reinforcing metal without rigid intersections, of which hexagonal mesh netting is an extreme case. In this netting the metal does not actually get into tension until the concrete slab cracks and fails, when it acts as a suspender. This material, of course, is unsuitable for the purpose of reinforcement, but it illustrates a type, such as "crimped" reinforcement, which is actually sometimes used. Then, again, there are

Mr. Himmelwright. forms of bar reinforcement, such as what is known as the Roebling System B, with 2 by $\frac{3}{16}$-in. flat bars set on edge. The reinforcing bars in that position in the slab have considerable value as beams, and the actual strength cannot be even approximated to by any formula that gives fair results with a bar of square or round section.

This type of construction is one of the most fire-resisting flat-slab methods now in use, because the bars, extending deep into the concrete, are protected by it, and in that position do not become heated as quickly as when the center of section of the reinforcing metal is nearer the under surface. When any of the ordinary formulas are applied to this type of flooring, they scarcely give it strength enough to support its own weight, whereas actual tests have shown its strength to equal, and in some cases exceed, that of the average of equivalent square and round bar reinforcement used in the ordinary manner.

The great difficulty in evolving a rational theory of design is that the strength is not the only important feature of fire-proof construction; fire resistance and durability are even more important. Conditions which will produce the greatest strength generally result in the poorest fire resistance; that is to say, the farther away the reinforcing metal is from the neutral axis and the nearer it approaches the underside of the slab, the greater will be the strength; on the other hand, the nearer the reinforcing metal approaches the under side of the slab, the thinner will be the protective covering and the more quickly the reinforcing metal will be heated in a fire so as to become weak.

The question of durability is most important. Exposure to the elements, which is a necessary condition in practical building construction, always causes initial rusting. When the reinforcing metal is in the form of bars or rods of considerable size and section, the rusting is not important, and does not materially weaken the metal. When, however, the reinforcement is in the form of wire or small mesh made from light sheet-metal or similar material, the rusting affects the strength very substantially by reducing the effective section, and when embedded in wet concrete it continues to rust in rainy weather and until the building is roofed over. In extreme cases such reinforcing metal under normal conditions becomes weakened by oxidation, so that it retains only a fraction of the strength shown by the formula.

Density is another factor that has large importance in cinder concrete fire-proofing. When stone aggregates are used and the concrete is thoroughly rammed so as to eliminate all voids, the action of the concrete approaches more nearly to that of the solid stone of which the aggregates are composed, and it cracks much more readily when exposed to heat and sudden cooling than when the concrete is full of voids. This, in fact, is the principal reason that cinder concrete

develops higher fire resistance than any concrete with stone aggregates. It is impossible to eliminate all the voids in the cinder aggregate by ramming, as such voids are always present in the aggregates themselves, to a greater or less extent, even if the cementing material is free from them. Mr. Himmelwright.

Tests made by the writer conclusively demonstrated that the more porous the concrete the less likely it is to crack and fail under sudden and violent changes of temperature. Cinders crushed so as to pass a ¾-in. screen, made into concrete mixed in the proportion of 1 : 2 : 5, and simply spread over the forms and leveled and patted down with shovels in segmental arch construction, will develop ample strength to carry ordinary loads up to, say, 200 lb., with a safety factor of 5. The material manipulated in this way will be extremely porous, light in weight, and exceed in fire resistance any other material known to the writer, except possibly a concrete made from "tetzlonti," a porous lava rock which the writer has used in a number of buildings in the City of Mexico.

It is well known, of course, that density is desirable, as tending to prevent oxidation of the steel reinforcement, as oxidation takes place more rapidly where voids occur in the surrounding concrete; but this oxidation practically ceases when the concrete becomes perfectly dry after a building is enclosed and finished. This phase of the problem, therefore, is of minor importance, especially when the reinforcing metal is in the preferred form of rods or bars of considerable section.

The economic advantage of voids in securing lightness is much more important than the initial oxidation, but it has never received the attention it deserves. A concrete made and deposited as above described will weigh only from 70 to 75 lb. per cu. ft., as against 95 to 100 lb. when thoroughly rammed. This is a gain of 25% in the dead load, in favor of economy, and, as above stated, ample strength is also obtained.

From what precedes it will be apparent that certain qualities and conditions which result in improved fire resistance, both in the form of the reinforcing metal and in the concrete, tend toward weakness, so that we have two conflicting elements in the design of cinder concrete floors which will forever prohibit the development of a practical rational formula of general application.

An important feature of this whole problem is the fact that it is a mistake to legislate or inaugurate regulations that are restrictive and tend toward the development and improvement of only a few forms of construction. The error of this, perhaps, is more plainly illustrated in the regulations governing theater construction than in anything else. All the building laws at the present time specify refinements of the conventional design for theaters, and absolutely

Mr. Himmelwright.

prohibit other designs which might provide greater safety and result in larger seating capacity. It is always better, in preparing building codes and city ordinances, to specify results to be obtained rather than detailed specifications.

Although strength in cinder concrete floors is a most desirable feature, others, equally desirable, should not be needlessly sacrificed and discouraged by ill-advised regulations. To make all forms of construction comply with a rational theory would immediately eliminate some of the best methods, as it would be obviously impossible, with the same materials and at the same cost, to develop a superior fire-resisting construction which would show by formula a strength equal to the ordinary method with the reinforcing metal as near to the under side of the slab as the law would permit.

The writer is disappointed that more progress has not been made in the direction of economy and in improving the fire resistance of cinder concrete floors. Practically all the progress during the last 5 years has been in the direction of securing the maximum strength with any given reinforcement; the protection of the latter being controlled by building codes and regulations, the thickness of the concrete protection varying from ½ to 1½ in. in different cities. The general tendency in this one direction of securing strength, which is largely the result of existing restrictive codes, has no doubt discouraged and minimized improvements in other directions, and has accentuated the supposed desirability of rationalizing all forms of cinder concrete construction.

The best that can be done with the problem is to develop a theory and formula in accordance with the results of the usual and more common forms of cinder concrete floors, as proposed by Mr. Waite, and, in addition, make a separate provision for the determination by test of the strength of those special forms of construction to which the formula and rational theory do not apply. This course has recently been followed in the most up-to-date building codes, in order not to discourage or prohibit methods which aim at superior fire resistance, lightness, and other economies.

There is room for great improvement in the matter of test loading of floors. In the past this has not been done in a scientific manner, and the varying and conflicting results of many of the tests is largely the result of improper or so-called "fake" loading. This is a subject which in itself can well constitute a separate paper, and is so important that the writer trusts that some one who is well-qualified will give attention to it in the near future.

Another great stumbling block in cinder concrete construction is in the definition of what constitutes a good quality of cinders. There is widespread ignorance in regard to this material, the popular idea being that it is coal ashes from ordinary stoves and furnaces, which

contain from 50 to 75% of soluble sulphates and silicates in the form of white powder and comparatively soft whitish lumps. Mr. Himmelwright.

Those who have this idea of cinders sometimes specify that they shall be screened, discarding all fine material, hoping in this way to eliminate the fine white ash, and, by retaining only the coarser portion, they expect to obtain suitable material. Ordinary coal ashes, if treated in this way, will still contain a large proportion of the soluble sulphates in coarse lumps.

Cinders with any considerable quantity of soluble sulphates are entirely unsuited for concrete aggregates, and should be prohibited. A good quality is obtained from large steam boiler plants in which the cinders are cooled by spraying. They contain practically no soluble sulphates or silicates, and from 95 to 98% of the fine material is grit fully equal to sand of good quality for the purpose of making concrete. When a building law requires screening, it can readily be understood that excellent material (the fine grit) is thus prohibited and wasted, which involves useless expense and a large loss in economy of construction. Locomotive cinders—on account of the coating of soot—and cinders from gas plants, are also unsuited for concrete. The extremely coarse clinker from bituminous coal is objectionable, as is, in fact, any coarse material.

To get uniform and satisfactory results, all cinders should be passed through rollers or crushers which will reduce them to a size that will pass a ¾-in. square-mesh screen. If sand is added to the fine material, so as to make a mixture in the porportion of 1 part of high-grade Portland cement to 2 parts of fine material and sand, and 5 parts of crushed cinder aggregate, a most excellent fire-resisting concrete will result.

A good definition for a suitable quality of cinders in a building code is the following:

"Cinders, where referred to in this code, shall be construed to mean the residue of coal consumed in large steam boiler plants, free from soluble sulphates and silicates, and in which the fine material is composed wholly of grit. (Note:—Ashes or cinders from ordinary coal stoves, ranges, hot-air furnaces, small steam boilers, or locomotive or gas-plant cinders, shall in no case be used for aggregates for cinder concrete.)"

An imaginary difficulty with cinders is the large quantity of unburned coal they are supposed to contain. Some years ago, in the writer's experience, work on one of the largest buildings in New York City was suddenly stopped by the architect. His inspector happened to be in the basement one day and saw a considerable quantity of black particles in the cinder. Cinders of the same quality had been used on the entire job, which was then half completed. The foreman in charge of the fire-proofing endeavored to

Mr. Himmelwright. satisfy the inspector by explaining that not all the black particles were unburned coal, but the inspector insisted that the work be stopped forthwith, and the architect so ordered. The inspector then selected a quart of the black particles from the cinder pile, took them to his own chemist, and had an analysis made. The result was that only 7% of the particles selected were found actually to be coal, and 93% consisted of stone, slate, and other impurities resembling coal. Although there appeared to be as much as 10% of unburned coal in the cinder supply, this analysis proved that there was in reality less than 1 per cent.

The foregoing are a few special features and conditions which the writer has noted in his experience in connection with cinder concrete fire-proofing. He has not attempted to go into the matter thoroughly, or in any detail, as he has not the time or opportunity to do so; but attention is called to these features of the problem in order to indicate that it is larger and more complex than is realized by the average person, especially by those whose experience is limited to a few tests made with the ordinary forms of reinforcement, and who consider only the question of strength.

Mr. Marburg. EDGAR MARBURG, M. AM. SOC. C. E. (by letter).—In the writer's judgment, the validity of the author's contention, that a design in which cinder concrete is utilized should be governed by rational formulas and prescribed working stresses for the materials involved, cannot successfully be brought into question, provided the design itself is of such a character as to admit of satisfactory analysis, and the requisite empiric data for the unit stresses are available. In the consideration of this important, though essentially simple matter, the discussion should not be befogged by the introduction of irrelevant issues, such as the admittedly variable character of the material and the unsatisfactory conditions as to inspection by which its use is frequently, if not usually, attended.

Inadequate inspection cannot reasonably be urged in support of adherence to unscientific methods in relation to the use of this material any more than in relation to any other material. Scientific and therefore economic design is, broadly speaking, impossible in any field of construction without rigid inspection in its execution. If the actual loads on building floors approximated more nearly to those prescribed by building codes, the many failures that would probably have resulted would long ago have enforced the lesson of adequate inspection. That circumstance, however, has no bearing whatever on the merits of the basic question under discussion. It may be, and probably is, true, that cinder, no matter how carefully inspected, is a much more variable product than stone or gravel. If so, that circumstance should be recognized by the selection of correspondingly conservative working

stresses in accordance with a principle as old as engineering itself. Such working stresses having been fixed, designs susceptive of rational analysis should be based on suitable formulas and not on tests, except in so far as tests may be made to establish the reliability of the premises —both as to theory and working stresses—governing the design, so that these may then be applied with confidence to other designs of the same general character, but differing as to dimensions, and as to the relative quantity and distribution of the material. To depend wholly on tests, under the conditions stated, would mean a reversion to empiricism unworthy of serious consideration in modern engineering. This statement is just as applicable to cinder concrete as to any other material of construction. Mr. Marburg.

If, on the other hand, the proposed design of which cinder concrete is to be an element is not demonstrably susceptive of rational analysis, only two courses are open: either to prohibit its use, or to have recourse to tests. That statement, too, is just as applicable, however, to stone or gravel concrete, or to any other material, as to cinder concrete. An attempt at differentiation in this connection at the expense of cinder concrete is so manifestly unwarranted that arguments to the contrary would seem superfluous. If such indeterminate designs are to be used, precisely the same precautionary considerations are applicable to cinder concrete as to any other material used under such conditions, namely, that the same care as to the selection of material, workmanship, and inspection should be applied to the construction itself as was applied to the particular elements subjected to test. Furthermore, the information yielded by such tests cannot safely be applied to other similar designs varying materially from those under which the tests were made, as to general dimensions and relative quantities and distribution of material,—that being the well-recognized limitation of empiric, as distinguished from rational methods.

To sum up, at the risk of reiteration, there is, in the writer's judgment, no inherent ground for making any distinctions whatsoever between the general considerations governing the design of reinforced concrete, whether the aggregate be stone, gravel, slag, or cinder. Whether the use of cinder concrete should be permitted where the strength element is an important factor; whether the prescription of proper working stresses would tend to militate against the economic use of this material; and whether its use, at least under certain conditions, should be avoided by reason of the tendency of the embedded steel to corrode, are questions with which the writer is not at present concerned. These questions have no bearing, however, on the fundamental principles which he has endeavored to emphasize, and their discussion in that connection would only tend to becloud the author's basic contention, which, in the writer's judgment, is incontrovertible.

Mr. Waite. GUY B. WAITE, M. AM. SOC. C. E. (by letter).—The purpose of this paper is:

1.—To show that cinder concrete is being used extensively in engineering construction;

2.—To present the fact that, in many cases, this material is not being used in accordance with well-defined engineering principles; and

3.—To obtain discussion among engineers with a view of either having its use included in the category of engineering, or of finding the reason that it should not be so placed.

So far as the writer is able to comprehend the discussions, nothing has been produced to give reasons for not designing cinder concrete according to unit stresses, as required of stone concrete.

The writer agrees with Mr. Marburg that no successful argument can be made for the arbitrary use of reinforced cinder concrete. His position, as head of an institution of engineering, and his connection with the American Society for Testing Materials, entitles him to recognition as an authority on this subject.

The only substitute for unit stresses suggested by any who have discussed the paper was a recommendation for more extensive testing. Of course, with sufficient testing to determine the proper working stresses, it would not be engineering to continue to test for every possible variation in spans, thickness of slabs, and difference in loadings.

Mr. Strehan makes several criticisms, among which are the following:

1.—That "the writer fails to find any plausible basis in the paper on which to establish constants for calculation and unit stresses for design;"

2.—That tension in the cinder concrete is omitted;

3.—That the difference between the conditions of cinder concrete between steel beams and the monolithic condition of all the reinforced concrete is not shown by the writer;

4.—That the trussing effect of side arches on the carrying capacity of any particular arch was overlooked in the paper; and

5.—That the continuous action of reinforcement anchored over the steel beams was ridiculed.

The matters referred to in Criticisms 3, 4, and 5 were mentioned on pages 1775 and 1783, and were evidently overlooked by Mr. Strehan. This was the explanation given for the great loads carried by the test arches.

In answer to Criticism 1, the writer would refer to Table 2 giving the unit stresses adopted for this material by several large cities which

have placed it on an engineering basis. There is also presented the first of a series of tests made by the Bureau of Buildings of New York City and Columbia University. Further, the writer has his own views regarding its proper working stresses; however, it was not the purpose of the paper to open up side issues for critics to utilize in misleading readers from the real purpose, namely, the establishment of some kind of unit stresses. Mr. Waite.

In his official connection with the Bureau of Buildings of New York City, Mr. Strehan took part in the tests recorded in Table 2. He refers in his discussion to a coming paper in which he will probably give more up-to-date information on the properties of cinder concrete than is now available. Therefore, it should become his duty as a public official—not the duty of the writer—to assign the exact proper units to be used for New York City. The writer's paper was on the general subject, and the tabulations cover almost every possible assumption.

As to Criticism 2, the writer pleads guilty of the crime of failing to consider tension in the concrete of the reinforced cinder concrete slabs. The report of the Special Committee on Concrete and Reinforced Concrete, which was referred to on pages 1797 and 1798, does not recognize tension, even in stone concrete. The stresses developed by the approved loads on cinder concrete, shown on those pages, make such an allowance (or even the talk about it) ridiculous.

As to Criticism 3, it was pointed out that the cinder concrete between steel beams was substantially cut in two and could not act in the certain and continuous manner of monolithic reinforced stone concrete; and for that reason, if for no other, greater stresses should not be allowed on the former than on the latter.

Referring to Criticism 4, Mr. Strehan admits that the outside spans of floor-slabs have not the same capacity as interior slabs, that is, those which have adjoining construction on both sides. As mentioned in the paper, the adjoining slabs thrust against the interior slab. Now, in the test slabs of the constructions discussed in the paper, Figs. 3 to 7, there were adjoining slabs; but in a large proportion of the slabs in buildings, one side (the outer side) has no slab adjoining to act against thrust.

In discussing Criticism 5, the writer must refer to experiences he has had with light reinforcing members used in the same manner as the wire mesh for the purpose of securing a continuous reinforcement. In 1900 he began the erection of reinforced concrete. His first contract was for cinder concrete floor-slabs located (as generally used in hotel and apartment-house construction) on the lower flanges of the steel supporting beams. The reinforcement consisted of small thin steel bands drawn continuously over the steel beams and bent down in the central part to reinforce the bottom of the concrete slabs. The

Mr. Waite. metal was kept straight and tight, with the object of securing a continuous action. It was found that in this condition the tamping of the concrete jarred the reinforcement and jerked it in the adjoining slabs to such an extent that the bond was disturbed and cracks appeared in the concrete over each piece of metal.

After trying out the system thoroughly, it was abandoned because no practical way of making it continuous could be found. The manner in which the wire mesh and the light reinforcement have been and are being used for the foregoing conditions may be described as follows: The wire or other metal is drawn over the steel beams and then stamped down on both sides and left with sufficient slack to prevent the jerking of the reinforcement, previously mentioned.

An examination of the work actually done in buildings will show much of the reinforcing wire, referred to by Mr. Strehan as being anchored over the beams, simply lying loose on top of them. In this condition there cannot possibly be any continuous action, because the strain comes on the center of the slab before the metal over the supports is drawn tight. All that can be expected of the reinforcement anchored over the beams, is to hold the slab suspended after it has broken in the center. Of course, in a single test slab, sufficient time and pains can be taken to insure continuous action and obtain illusionary results.

Mr. Strehan does not agree with the writer that one kind of reinforcement is as good as another. The writer would modify this statement, to withstand criticism, by saying that what was really meant was that, any forms of steel sufficiently embedded in concrete for the adhesion to develop the full section of the metal will give substantially the same results. This fact was shown for stone concrete in tests by A. N. Talbot, M. Am. Soc. C. E., several years ago, and the tests in Table 4 for cinder concrete seem to show the same result.

The writer referred to the subject of the capacities of various kinds of reinforcement because the arbitrary manner of using cinder concrete has led to all kinds of misrepresentations by sellers of reinforcements. Some have obtained approvals with much less reinforcement than others; all make claims of some peculiarity in their form or kind of material, which makes it superior to all others. In the proposed Building Codes of 1912 and 1913 of New York City (Table 1), this recognition of superiority was to have been specified in the law.

On account of the seeming protection, by the arbitrary use of the construction, large industries have been built up. It is the writer's belief that some of these industries have been the father of many of the agitations for new building codes.

Several who discussed the paper referred to the corrosion of steel embedded in cinder concrete. Some seem to think cinder concrete

should be restricted, because there is no certainty that the steel is protected against corrosion. As the situation stands, these arguments for restriction are too late; cinder concrete is already used extensively, and the object of this paper is to have it used in a rational manner. Mr. Waite.

The subject of the corrosion of steel, however, is vital, and should be better understood. It is the writer's experience that the average quality of hard coal cinders mixed in the proportion of 1:2:5, made wet, and tamped thoroughly, will sufficiently coat the surface of the steel with neat cement wash to protect it thoroughly and permanently from corrosion. When the grade of cinders runs too coarse to fill the voids, the concrete is porous, in which case, unless the steel is first coated with cement wash, the concrete is likely to contain air holes extending to the bare steel.

Mr. Himmelwright regrets the fact that the writer did not include the discussion of other forms of cinder concrete than the ordinary plain reinforced construction. The construction discussed in the paper is that which has automatically displaced those referred to by Mr. Himmelwright, and—perhaps it is not exaggeration to say—is the form of construction in universal use, where competition is allowed. The writer may lament this as much as Mr. Himmelwright, but, knowing the facts, he did not wish to confuse the subject by the introduction of constructions so far removed in relationship that it would be useless to attempt to make a formula to include them all.

In defence of the writer's rational treatment of the subject and of the general tabulations, to cover the use of almost all constants, he begs to state that this was done principally because, in past discussions, before aldermanic committees, etc., the unit-stress idea has been ridiculed, and more than one member of the Society has scouted the idea that a rational treatment was practical.

It is hoped that engineers will awaken to their own interests by insisting that cinder concrete, when used as an engineering construction, shall be elevated from the field of irresponsible contractors, referred to by several in their discussions, to the field of Civil Engineering.

MEMOIRS OF DECEASED MEMBERS

Sir WILLIAM HENRY WHITE, Hon. M. Am. Soc. C. E.*

Died February 27th, 1913.

William Henry White, the youngest child of Richard and Jane (Matthews) White, was born at Devonport, England, on February 2d, 1845. He attended the local schools until March, 1859, when, at the age of fourteen, he was apprenticed at the Royal Naval Dockyard. In addition to his practical work, he attended the Dockyard School, maintained by the Admiralty as a part of the technical training of its apprentices, where he showed his ability by winning an Admiralty Scholarship in 1863.

It was during this time that propulsion by steam was being substituted for sails, and sailing vessels were being reconstructed as screw-propellers. The use of iron in place of wood for the hulls of vessels was also being introduced, and young White had the advantage of combining practical work with the preparation of designs, the results of which were clearly shown in his highly responsible work of later years.

In 1864, the Admiralty, at the urgent request of the Institution of Naval Architects, established the Royal School of Naval Architecture at South Kensington, in connection with the Science and Art Department. This school, which was instituted in order that students in naval architecture might take advanced courses in that science, was afterward merged into the Royal Naval College at Greenwich. Sir William was one of the first students received at this newly founded college, taking first place at his entrance examinations in 1864, which standing he maintained during his three years of attendance. He was graduated in 1867, with the highest honors, as a Fellow (first-class). In 1870 he was made Professor of Naval Architecture at this school, which position he held until 1881. This work was in addition to his duties at the Admiralty. Among his pupils during this period were men who afterward became chief constructors and naval architects in the various navies of the world, and it was largely due to his influence that the school's high standing was acquired and maintained.

Immediately after his graduation from the Royal School of Naval Architecture, Sir William entered the Admiralty as Private Secretary to Sir Edward Reed, Chief Constructor, being engaged in the solution of scientific problems in naval architecture, etc. It was while he occupied this position that he was brought directly in touch with the design of armored vessels which were then a new invention, thus adding to his experience as a designer and constructor.

* Memoir prepared by the Secretary from information on file at the Society House.

In 1870, on the resignation of Sir Edward Reed and the appointment of a Commission to continue his work, of which Sir Nathaniel Barnaby was Temporary President, Sir William was made Professional Secretary to the Commission. In this position, and with the aid of his life-long friend, the late Mr. William John, he had charge of, and carried out, numerous experimental inquiries in regard to the stability of ships for the Commission appointed to investigate the capsizing of the ironclad *Captain,* in the Bay of Biscay. The results of these inquiries were embodied in a paper* on the subject presented before the Institution of Naval Architects, which greatly advanced the science of ship design.

In 1875, Sir William was promoted to the rank of Assistant Constructor, and, in 1881, he was advanced to that of Chief Constructor. While in this position he effected the organization of all the trained architects in the Admiralty into one corps—the Royal Corps of Naval Constructors—which organization has proved of great service to the British Navy. In 1883, when Sir William Armstrong was establishing his great shipyard at Elswick, England, he offered to Sir William the difficult feat of organizing and directing the Warship Building Department. While in this position he made designs for naval vessels for Austria, Italy, Japan, China, and Spain, and had charge of the construction of several vessels for the British Navy, among which was the *Victoria* which, in 1893, was sunk in the Mediterranean by a collision, with great loss of life.

In 1885, Sir Nathaniel Barnaby resigned his position at the Admiralty on account of ill-health, and on his recommendation, Sir William, having been released from his contract with Sir William Armstrong, Mitchell and Company, was appointed Director of Naval Construction and Assistant Controller of the Navy, which position he held until February, 1902, when he also was compelled to resign on account of ill-health.

Sir William began his work as Director of Naval Construction at the Admiralty at the beginning of a period of expansion in British Naval affairs. Prior to that time no uniformity in the vessels of the fleet had been attempted, and the reorganization of the administrative and operative departments was demanded. The Northbrooke programme, which comprised the carrying out of the construction of a number of new ships and the rapid completion of those already begun, was his first task, and the work was accomplished so expeditiously that when the Naval Defence Act of 1889 was passed, providing for the construction of 70 ships at a cost of £22 000 000, sterling, Sir William had a chance to carry out his idea of homogeneity in a fleet, namely, ships bearing a distinct relation to each other and to the

* "The Calculation of the Stability of Ships, and Some Matters of Interest Connected Therewith," by W. H. White and W. John, *Transactions*, Inst. of Naval Archts., Vol. XII p. 77.

fleet as a whole. The Spencer programme of 1894 and the Goschen programme of 1896, the latter comprising the construction of the first dreadnought of the British Navy, were also carried out under his supervision. The development of the torpedo boat and the torpedo-boat destroyer was due to his skill, and he also designed the river gun-boats built for the Nile Expedition under Lord Kitchener. When he retired in 1902, he had had responsible charge of the design and construction of 245 vessels, valued at about £100 000 000, sterling. All this vast work was done under constantly changing conditions of material, type, size, speed, armament, etc., and to him may be attributed the introduction of several innovations in British warships, notably that of water-tube boilers and the use of oil fuel for firing boilers. He had entered the Naval Service before the first real ironclad was ordered, and had seen the Navy develop from one of wooden ships to the present-day steel dreadnought.

He was awarded a C. B. in 1891 and a K. C. B. in 1895, and on his retirement from the Admiralty in 1902, Parliament voted him a special grant for "exceptional services to the Navy."

After a rest and the recovery of his health, Sir William began practice as a Consulting Naval Architect, and was engaged on many important works. He was a member of the Cunard Commission which decided the type of machinery for the *Lusitania* and the *Mauretania;* a Director of the firm of Swan, Hunter, and Wigham Richardson, Limited, the builders of the *Mauretania,* during her construction; a Director of the Parsons Marine Turbine Company; and a Director of the Grand Trunk Railway Company after that Company became the owner of steamships. He designed steamers with geared turbines for service between India and Ceylon, and was a Member of the Government Commission to investigate the question of load lines of merchant ships.

On February 27th, 1913, Sir William suffered a paralytic stroke at his offices in Westminster, and died the same day at Westminster Hospital to which he had been removed.

He was twice married, his first wife having been Miss Alice Martin who died in 1886. In 1890, he was married to Miss Annie Marshall who, with three sons, all of whom are officers in the British Navy, and one daughter, survives him.

Sir William was a frequent contributor to technical and engineering journals and to the publications of technical and scientific societies. He was also the author of several books, his "Manual of Naval Architecture" and "Treatise on Shipbuilding" having become classics, the former being translated into German, Italian, Russian, and Spanish. His most notable contributions were made to the *Transactions* of the Institution of Naval Architects, the best known being the papers on the stability of ships (already referred to), the rolling of sailing ships,

and the effect of bilge keels on rolling, and his description of the design of the battleship *Royal Sovereign*. That his versatility was not confined to engineering, but extended over a wide range of subjects, is shown by his contributions to various newspapers and magazines.

He was always greatly interested in questions of engineering and technical education, owing, perhaps, to the difficulties he experienced in acquiring his own training. He was Chairman of the Committee on the Education and Training of Engineers, appointed by the Institution of Civil Engineers in 1903, and was also a member of the Governing Body of the Imperial College of Science and Technology.

Sir William was connected with many engineering and scientific societies and associations, to which he was always ready to give advice and assistance, and many of these had honored him with offices of distinction. He was a Fellow of the Royal Society; Honorary Vice-President of the Institution of Naval Architects; Past-President of the Institutions of Civil Engineers, Mechanical Engineers, Marine Engineers, Junior Engineers, and the Institute of Metals; President-Elect of the British Association for the Advancement of Science, having been President of the Mechanical Science Section. He was also an Honorary Member of many other British and foreign technical societies, including the American Society of Mechanical Engineers and the Society of Naval Architects and Marine Engineers.

In 1911 he was awarded the John Fritz Medal by a Board of Award appointed by the four leading American engineering societies—Civil, Mining, Mechanical and Electrical—for "notable achievements in naval architecture." He had also received honorary degrees from many colleges and universities, among which were LL.D. from Glasgow, D. Sc. from Cambridge, Durham, and Columbia (New York City) Universities, and D. Eng. from Sheffield. He also belonged to the Athenæum and British Empire Clubs.

In regard to his personal characteristics, the following* was written by one who had been on terms of close friendship with him for more than 30 years:

"He was beyond compare the straightest and most generous man alive. He was, without exception, the most remarkable man I have known, and as honest as the day. From the start he had never a soul to help him, but achieved distinction by his own ability and industry. A foreign government made overtures to him to superintend the total reconstruction of its then somewhat infirm war flotilla. White, with that fine sense of honour which was inseparable from the man, refused a most lucrative offer because he considered the value he could return for the great monetary payment held out to him, could only have been acquired by his intimate knowledge gained in the service

**The Times* (London), February 28th, 1913.

of our Admiralty. The amount of work that he did without repayment in those educational institutions which are bringing forward the most intelligent men of the new generation, was beyond compare. His self-negation, accurate knowledge, and charming manner extraordinarily enhanced by a great gift of description, made him such a professor of scientific and applied mechanics as the world has seldom seen."

Sir William Henry White was elected an Honorary Member of the American Society of Civil Engineers on December 16th, 1904.

ERNEST PONTZEN, Cor. M. Am. Soc. C. E.*

DIED OCTOBER 13TH, 1913.

Ernest Pontzen was born at Budapest, Austria-Hungary, on January 20th, 1838. He obtained his early engineering education at the Polytechnic School of Vienna. In 1856, he went to Paris and entered the French engineering school, l'Ecole Nationale des Ponts et Chaussées, as an "Externe" student, and was graduated in 1860 at the head of his class.

After a study trip in England, Mr. Pontzen returned to Austria-Hungary, and filled various positions in connection with the railways and harbors of that country. He was Consulting Engineer to the Anglo-Austrian Bank for 4½ years, and, in that capacity, directed the construction of several railway lines, made a study of various railway projects, and became the Director of a railway company in Hungary.

In 1873 he made his first study trip to the United States. He made a second trip in 1876, to serve as member of the Jury in the Railway Group at the Philadelphia International Exposition. Returning to Paris, he established himself there permanently, and eventually became a naturalized French citizen. He married Miss Hirtz, daughter of the distinguished physician of that name, professor of the University of Strassburg, who survives him.

Mr. Pontzen was the author of numerous publications dealing with the construction and operation of railways. His most important work, issued in collaboration with Mr. Lavoinne, and entitled "American Railways," was published in 1880-82, the first volume dealing with the construction and cost of railways, and the second with their operation.

Mr. Pontzen served on various important commissions. He was a member of the Government Commission for the technical exploitation of railways since 1884, which was almost from its inception. Acting in this capacity, he was one of the delegation which represented the French Government at the International Conference for the promotion of technical uniformity on railways, and more recently he was a delegate of the Minister of Public Works to the International Railway Congress at Washington, and, later, to that at Berne.

He was selected as expert and arbitrator in very important cases, such as the Loetschberg Tunnel, the Harbor of Varna, the acquisition of the Algerian East, and the Cé Bridge accident.

He was a Member of the Council on Instruction (Conseil d'In-

* Memoir prepared by Charles L. Strobel, M. Am. Soc. C. E.

struction), and of the Council on Betterment (Conseil de Perfectionnement) of l'Ecole Nationale des Ponts et Chaussées.

Mr. Pontzen was a Past-President of the Society of Colonial Engineers and Past Vice-President of the Society of Civil Engineers of France. He was one of the founders and, up to the time of his death, was President of the Friendly Society of "Externe" Graduates of l'Ecole Nationale des Ponts et Chaussées. He was also an officer of the Legion of Honor and a Commander of various French and foreign orders.

This brief outline of Mr. Pontzen's career will indicate that it was fruitful in engineering achievement and rich in well-merited honors. He went to France as a stranger, and died in the country of his adoption, having won not only the esteem of Frenchmen, but to a singularly high degree their love and affection.

Mr. Pontzen spoke French, German, English, Italian, and Hungarian, and was able to converse with the foreign engineers he met at Congresses in their mother tongue. All who met him in this way remember him most pleasantly for his uniform courtesy and cheer. American engineers who had the good fortune to know him were always sure of a cordial welcome at his hands, and he freely gave them of his time and attention in the promotion of their interests or their comfort and pleasure. He was the kindest, most considerate, and most devoted of friends. It may be said of him that the conduct of his life was so noble and inspiring, that the highest tribute will be paid to it that can come to any man, in that it has left the desire in the younger men who knew him to emulate his example as best they can.

Mr. Pontzen was elected a Corresponding Member of the American Society of Civil Engineers on January 5th, 1876.

ARTHUR LINCOLN ADAMS, M. Am. Soc. C. E.*

Died September 17th, 1913.

Arthur Lincoln Adams, son of Jacob Clendennin and Nancy McCoy (Hamilton) Adams, was born on September 15th, 1864, on a farm near Greensburg, Ind. He was a lineal descendant of John Adams, who came to Plymouth Colony, Massachusetts, in the ship, *Fortune*. in 1621.

When about eight years of age he moved with his parents to Topeka, Kans., where the family resided for a period of about three years, and then returned to Indiana. In 1882, soon after the death of Mr. Adams' father, the widow again moved with her children to Topeka, where she resided during the remainder of her life.

In his early boyhood Mr. Adams attended the district schools of Indiana, and the public schools of Topeka. Later, he entered the Greensburg High School, from which he was graduated in 1882. He was a student at Hanover College, Ind., in 1882-83, and Washburn College, Topeka, Kans., in 1883-84.

During the summer of 1883 he was in the employ of the County Surveyor of Shawnee County, Kansas, and assisted in surveying in and about Topeka. During the following summer he was with a surveying party engaged in subdividing Government land in Southwestern Kansas. Starting as a Chainman, he was promoted to Transitman before the close of his engagement. When the survey was completed, each member of the party was requested to state, under oath, that the work had been done correctly, to the best of his knowledge and belief. Being somewhat in doubt as to the accuracy of the survey, Mr. Adams, together with several others of the party, strenuously objected to being sworn. At this juncture, one of the youthful mutineers made the discovery that Quakers were permitted to affirm in lieu of taking oath, whereupon Adams insisted that the same privilege be accorded him and his conscientious associates. His arguments prevailed, the affirmations were duly made, and all the members of the party were given honorable discharges.

In the fall of 1883 students of the State University of Kansas started a movement to organize a State oratorical association, to affiliate with the Interstate Association. Mr. Adams was a delegate from Washburn College to the constitutional convention. He took a prominent part in the proceedings, which resulted in the organization of a State association, and was active in arranging for the first contest. He participated in the trial and settlement of a famous case of plagiarism which developed in that contest.

* Memoir prepared by Franklin Riffle and W. A. Cattell, Members, Am. Soc. C. E., and A. Kempkey, Assoc. M. Am. Soc. C. E.

In 1884 he became a student in the Engineering Department of Kansas State University, and was graduated in 1886. At a public rhetorical recital he read a humorous essay entitled "The Philosophy of Selfishness", in which his theories and conclusions were so extreme as to create a sensation among his hearers. Only a few of the audience appreciated the undercurrent of humor which pervaded the essay, or realized that the extreme views advocated by the speaker were purely for rhetorical effect.

During the spring of 1886, he obtained employment in the Engineering Department of the Burlington and Missouri River Railroad, in Nebraska, in the office of one of the resident engineers, and was soon promoted to Division Engineer on Construction. In this position he displayed unusual efficiency, originality, and judgment, for one of his age and experience.

During the summer of 1887, he concluded to try his fortune on the Pacific Coast. He accordingly accepted a position as Leveler in a location party on the Oregon Pacific Railroad. When the party was disbanded, during the following winter, he went to Southern California in search of employment and spent several months in Los Angeles.

During the spring of 1888 he returned to the Pacific Northwest to accept a position as Assistant Engineer on Construction, offered him by the Chief Engineer of the Oregon and Washington Territory Railway, now a part of the Northern Pacific System. In this position he had an excellent opportunity to observe and study the wonderful resources of Eastern Oregon and Washington. He was quick to grasp the significance of the development of this vast inland empire, and to see with prophetic eye the varied and multitudinous engineering problems to which this development would give rise. Many thriving towns were in various stages of development, and some of them had already reached a point where the necessity for sewer and water systems was occupying the attention of their citizens.

Up to this time, Mr. Adams, like many other young engineers, had followed the lines of least resistance. He had become a railroad engineer through force of circumstances. After graduation he had accepted a position on a railroad because it was the only position immediately available. Municipal engineering, however, was his choice, and he now determined to make a start in this field. The Town of Pendleton, Ore., had recently created the office of "City" Engineer, and the Mayor was given the appointing power. Before resigning his position as railroad engineer, Mr. Adams discussed the matter with the Chief Engineer, who accepted his resignation with regret, but, nevertheless, co-operated with him in his efforts to secure the coveted municipal position. Mr. Adams was successful in obtaining the appointment, at the same salary he had received from the railroad. In conducting the negotiations which terminated so happily, he exhibited

that remarkable business sagacity which contributed so largely to his later successes. Foreseeing that as "City" Engineer of a small town his duties would not be very onerous, he stipulated that he should be granted the privilege of performing such private work as he might see fit to undertake during his term of office. He prepared a contract fully covering this phase of the question, and it was duly signed by the contracting parties. It was not long, however, before his private practice assumed such large proportions that he found it advisable to resign.

In 1890 Mr. Adams prepared plans and specifications for a system of water-works for the Town of Dayton, Wash., and superintended its construction. During the following year he performed a similar service for Colfax, Wash.

In the summer of 1891 he entered into partnership with his brother-in-law, Robert C. Gemmell, M. Am. Soc. C. E., under the firm name of Adams and Gemmell. The new firm designed and superintended the construction of water-works for La Grande, Ore., and Waitsburg, Wash. They also designed a water system for Heppner, Ore., and a water-power plant at Kalama, Wash., neither of which was constructed, however, owing to the financial panic of 1893. The firm did considerable miscellaneous surveying in Oregon and Washington, including two large irrigation canals on the Umatilla River, Oregon, one on the Walla Walla River, Washington, and one on Eagle Creek, near Baker City, Ore. The construction of these canals was indefinitely postponed by the "panic". They prepared plans and specifications for a water-works system for Athena, Ore., and a sewer system for Montesano, Wash., both of which were constructed, although not under their supervision.

During the fall of 1893 Mr. Adams made a very complete report on an increased water supply for Astoria, Ore., and recommended the construction of a new system. His report was adopted in its entirety by the Board of Water Commissioners, and Adams and Gemmell were authorized to make the necessary surveys and plans. Before the completion of the preliminary surveys, the partnership was dissolved, and the work was continued by Mr. Adams, who, as Chief Engineer, carried it through to a successful termination—less than 3 years after having made his preliminary report. An admirable description of this work was written by Mr. Adams, under the title, "The Astoria City Water Works".* This paper won for its author the Thomas Fitch Rowland Prize for 1897. In his paper he described quite fully an ingenious device (of his own invention) for opening and closing two fire gates from a central station; this being accomplished "by opening and closing

* *Transactions*, Am. Soc. C. E., Vol. XXXVI, p. 1.

an ordinary stop-cock at the end of a line of ordinary ¾-in. service pipe leading to a patented governor attached to each of the gates."

During the construction of the Astoria Water-Works (1895), Mr. Adams prepared a report on a new water system for Eureka, Cal., and on the character and probable cost of the private works then supplying the city. After the completion of the Astoria Water-Works, he opened a consulting office in San Francisco. During 1897 he prepared plans and specifications for the reconstruction of water-works at Centralia, Wash., and also reported on projected new works for Lakeport and Oceanside, Cal.

From 1897 to 1900, he held the dual position of Manager and Chief Engineer of the West Los Angeles Water Company and the West Side Water Company. During this period he resided in Los Angeles, where he also carried on a consulting practice. As Chief Engineer of the two Los Angeles Water Companies, he was called on to plan and construct extensions in order to keep pace with the steadily increasing demand. He constructed a 5 000 000-gal. reservoir, and a pipe line to the Soldiers' Home, Santa Monica. In connection with the pipe line, he designed an ingenious weir and gate device for maintaining a definite delivery under varying conditions of pressure.

During this period Mr. Adams filled various engagements, among which were the following:

He was a member of a commission of engineers engaged by the City of Los Angeles to determine the value of the improvements made to the water-works of that city during the 30 years' lease by the Los Angeles City Water Company; he was a member of a commission of engineers engaged by the City of Pasadena to appraise the several water-works used for the supply of that city; in conjunction with J. B. Lippincott, M. Am. Soc. C. E., he reported on sources of supply and plans for an entirely independent system of water-works for Pasadena; he developed a water supply and prepared plans for its utilization, in the irrigation of a large ranch near Corona, Cal.; he prepared a report outlining plans, and giving estimates of cost, for a combined municipal water supply and power installation for Missoula, Mont.; he prepared plans and specifications for the reconstruction of the water-works of Pendleton, Ore.; he reported on the probable effect on the water supplies of the San Gabriel River, California, of the contemplated construction of certain power plants on that stream; he prepared a report on the value of the properties of the West Los Angeles Water Company and the West Side Water Company, as an aid in effecting a sale of these properties to the City of Los Angeles; he prepared a report and gave evidence concerning the value of the properties of the Contra Costa Water Company used for supplying the City of Oakland and its vicinity; and he prepared a report and gave evidence concerning the causes of an extensive land-slide which had for some years prevented

the use of the large reservoir on the west side of the Willamette River, at Portland, Ore.

In 1901 Mr. Adams became Engineer and Manager of the Contra Costa Water Company, Oakland, after having been retained as an expert in the important litigation case which had just at this time been concluded.

The case was the now famous one known as the Hart case, and was instituted to enjoin the City of Oakland from enforcing the rate ordinance enacted by the City Council. Although several other eminent engineers were engaged in the case, it was conceded by all that the clear and convincing testimony of Mr. Adams as to values, and his clear exposition of the theories of valuation, had more to do with the victory of the Company than any other one thing. The Company was granted in effect an increase in rates amounting to 15%, and, although in later years engaged in many important legal matters, Mr. Adams often remarked that he considered this case the most important of its kind in his career.

Early in 1903, Mr. Adams resigned from the management of the Contra Costa Water Company and opened a consulting office in San Francisco, and from this time he continued steadily his consulting work. Although his practice was never such as to require a large organization, it was at all times of the highest type. Among the more important of his engagements may be cited that with the City of Victoria, B. C. His report dealt with the existing conditions and present and future needs, covering the subject in complete and exhaustive detail. Later, he prepared plans and specifications covering the then present needs, and, under his direction as Consulting Engineer, they were executed.

While still retaining his consulting practice, he was, in 1906, made Chief Engineer of the Department of Greater Water Supply of the Peoples Water Company, successors to the Contra Costa Water Company, supplying water to seven cities on the east side of San Francisco Bay. Complete preliminary studies and investigations were carried on by this organization looking to the more than doubling of the water supply of this Company. A complete comprehensive plan of future development was outlined, and construction was begun on the more important elements, but financial stringency prevented carrying out more than a small portion of this work.

Mr. Adams did a very large amount of valuation work, particularly of public utilities. Among the more important properties on which be reported may be mentioned the Peoples Water Company, the Spring Valley Water Company, the Palermo Land and Water Company, the Virginia and Gold Hill Water Company, the Petaluma Power and Water Company, and numerous others.

In 1909, he designed the irrigation system for the Patterson Ranch Company, at Patterson, Stanislaus County, California. This system provides a means for irrigating, by pumping, about 18 000 acres of land on the west side of the San Joaquin River in the San Joaquin Valley, and was at that time considered to be of unique design and the most extensive and complete system of the kind in the United States.

At the time of his death Mr. Adams was engaged in the construction of a plant similar in design to the Patterson Ranch system, but more elaborate as to details, for the irrigation of about 22 000 acres of land near Brentwood, Contra Costa County, California. He was also, at the time of his death, Consulting Engineer for the Peoples Water Company, and was engaged by this Company in important rate litigation pending in the Federal Court.

Mr. Adams was foremost among those who organized the San Francisco Association of Members of the American Society of Civil Engineers. He was elected Treasurer of the Association for a term of 2 years, and became its third President in 1907. Immediately after the earthquake in 1906, he was instrumental in holding a series of special meetings of the Association, which resulted in a study of "The Effects of the San Francisco Earthquake of April 18th, 1906, on Engineering Constructions," by a general committee and six special committees composed of members of the Association, followed by extensive reports.* Mr. Adams was Chairman of the Committee on the Effects of the Earthquake on Water-Works Structures, and wrote the Committee's report.

Mr. Adams held a high rank both personally and professionally. He was exceedingly democratic in his ideas, very simple in his tastes, and exceptionally companionable. He possessed a keen, analytical mind that served him well in the litigation with which he was so frequently connected as an expert witness. He delighted in the legal subtleties of these cases no less than in the engineering problems involved. Although active in assisting and supporting the attorneys with whom he was associated, he always commanded the admiration and respect of the engineers and attorneys who opposed him. His ability was acknowledged and his work commended by friend and foe alike. His professional career was one of uninterrupted achievement from its beginning to its untimely end. Long before his death he had attained a foremost place in the ranks of the leading hydraulic engineers of the Pacific Coast. He possessed in a marked degree the rare combination of professional skill and business acumen which enabled him to procure for his clients the best attainable results at a minimum cost. Having early in his career established a reputation for efficiency and integrity, he thereby created a demand for his services, for which, usually, he was able to name his terms. While always fully alive to his personal

* *Transactions*, Am. Soc. C. E., Vol. LIX, p. 208.

interests, he never for a moment overlooked the interests of his profession. He continuously sought to uphold its dignity, and to educate the public to a proper conception of its importance as a factor in the world's development. On numerous occasions he fearlessly exacted from his clients the recognition to which he believed his profession was entitled.

Mindful of his early struggles for recognition, he strongly sympathized with young engineers, and was ever ready to extend a helping hand to those in need of assistance. It was largely the desire to aid the younger members of the profession that induced him to take a prominent part in the organization of the San Francisco Association of Members of the American Society of Civil Engineers. He believed that they, more than the older members of the Society, would be benefited by membership in this organization. During the practice of his profession he gave employment to many young engineers. He invariably treated his subordinates with forbearance and tolerance, and thus secured and retained their respect, loyalty, and friendship.

Mr. Adams possessed a gift which, unfortunately, is quite rare among engineers—that of clear, terse, forceful expression. This applied to his speech no less than to his writing. Whether writing a report, explaining an engineering proposition to a client, elucidating a technical point while on the witness stand, or addressing an audience, he invariably chose his words well, and without hesitation. In argument he was convincing and rarely failed to carry his point. Whenever he took a stand for or against a proposition, he gave the impression of being master of the situation. He never allowed himself to be swerved from the course which his conscience or his judgment marked out.

His energy and ability, however, were not devoted entirely to his engineering practice. He was a zealous and efficient worker in the cause of religion and morality. He was prominent in the affairs of the church of which he was a member, and also gave much of his time and resources to the Young Men's Christian Association and Young Women's Christian Association of Oakland. The Y. M. C. A. Building, and the new First Presbyterian Church edifice, both in Oakland, are to a very large extent the results of his effective work.

Mr. Adams was a well-read man. He was especially fond of poetry. Among the poets, Robert Burns was his favorite. He occasionally wrote verse of a serious nature, although not for publication. One of the writers of this memoir has in his possession several of Mr. Adams' poems, one of which, entitled "A New Year's Revery", possesses exceptional merit.

Frank T. Oakley, M. Am. Soc. C. E., who was a classmate of Mr. Adams in the public schools of Topeka and in the State University

of Kansas, and also was associated with him during his professional career, writes of his life-long friend as follows:

"He was always very tolerant of the beliefs of other people, never seeming to wish to force his opinions upon others. However, when it came to argument, few could hold their own against him. My observation leads me to believe that he always followed a thoroughly digested plan of conduct and business from which he did not allow himself to vary in principle, and I believe much of his success in life's problems was due to this fact as well as to his unusually good judgment.

"Some time over twenty years ago, while sitting in camp, he outlined to me his idea of an engineering career leading up to a consulting business. When I came to live in California, I was impressed with the similarity of his outline of a career and his experience, and the way he carried on his professional business. He always insisted on giving any undertaking his best thought, and would not be identified with it otherwise, not sparing himself hard work; and as a consequence he was always accorded a leader's place by his associates.

"I believe that Arthur L. Adams was the finest character I have ever known, and that it is not possible for me to know how much I owe to association with him. His influence was always for good and manly things. This feature did not eliminate, or in any way curb the mischievous or boyish spirit, or the joy in sport or in a practical joke, but it did eliminate any meanness—a thing that was impossible in his nature.

"In our close association, including two years of living together, I never knew him to be gushingly confidential. There was always something of a reserve about him. Nevertheless, I never knew one in whom I could more freely confide, and who was more ready with the best and most kindly and tactful advice. I knew him best during the character-forming age, when boys are fully themselves and freely open to their playmates. Yet I never knew him to do an act or express a thought that was not in accord with his later high moral and mental attainments. Nor was he in the least prudish, nor did his associates ever think of calling him a 'goody-goody boy.'

"During our college days we frequently discussed all kinds of subjects pertaining to human life and affairs. Whether the subject was a social matter, the mathematics of gambling, or theology, freedom and frankness of thought and expression were always the same, always tending to a greater and more beneficial knowledge of the subject and, in fact, a crystallization of one's own opinions. When thinking of Arthur Adams' character, I am reminded of what Riley makes the farmer say of his friend and neighbor:

"'Fer the name of William Leachman and
True Manhood's jist the same.'"

C. Derleth, Jr., M. Am. Soc. C. E., pays the following tribute to Mr. Adams, both as a man among men and an engineer:

"Through his writings, particularly on wooden stave pipe, I have known Mr. Adams by reputation at least since 1896. I did not meet him personally until 1904. But since then I feel that I have had an

opportunity to know of his work and to appreciate his personality. For the last eighteen years I have worked for, or have met in various ways, a great many engineers whose names are prominent in our national societies, and these men hail from all quarters of the United States. I am convinced that among them none has combined in the highest sense, better than Mr. Adams did, all of those qualities which contribute to the professional and personal character of an engineering gentleman. One always associated with Mr. Adams the qualities of integrity, industry, wide reading, learning in fields other than engineering. He had a great command of the powers of speech. He wrote fluently. He was an engineer who used reason and who had powerful judgment. Above all, he was a man of the finest moral instincts. He was always ready to help improve the tone of engineers and to raise the standard of engineering ethics. Indeed, he applied the same qualities to wider fields and was a power for good and uplift in the community in which he lived. As more men like Mr. Adams become identified with engineering, so in proportion will the community's respect for the profession of engineering increase."

Augustus Kempkey, Assoc. M. Am. Soc. C. E., writes of Mr. Adams as follows:

"As his Principal Assistant for some eight years immediately preceding his death, the writer came to know Mr. Adams as a most refined and courteous gentleman of the highest type, every ready to give the best that was in him to the uplift of humanity and to the assistance of those around him. His thoughts were ever on the highest plane and his judgment, particularly in engineering matters, remarkable to a degree that at times seemed little short of uncanny. His mind, working perfectly clear and along direct lines, enabled him to grasp the essentials of an engineering problem almost at once, and his solution of these essentials was equally direct and expeditious. His criticisms of and orders to his subordinates were always given in a most kind and considerate manner, and no matter what the subject, he was always ready to receive suggestions and ideas from those directly under him. In all my association with him, I never saw him either reject or accept a suggestion without being fully informed as to its significance and value; and in the case of rejection, pointing out clearly his reasons therefor, and if accepting, giving due praise and weight to the value thereof. In short, he was a man who inspired and encouraged ideas and suggestions from his subordinates, while withal he never provoked that familiarity which ultimately breeds contempt. In each of his reports it will be found that he never failed to give credit to those who had assisted him in their preparation.

"He believed the engineering profession to be one of the greatest, and maintained an ethical standard such as few can hope to measure up to. By the life and conduct of such men as he, the engineering profession will be raised to its highest plane, and the moral tone of all with whom they come into contact cannot fail to be enriched. With all who had the privilege of working with Mr. Adams, and with all who came into contact with him in any walk of life, his memory will live forever as that of a true gentleman and an eminent—or rather a pre-eminent—engineer."

Mr. Adams was married on December 18th, 1889, to Mary Gemmell, of Topeka, Kans., who, with two sons and three daughters, survives him. Robert, the eldest, is a student in the Engineering Department of Stanford University, and two of the daughters are students in the University of California.

Mr. Adams died at his home in Oakland, Cal., on September 17th, 1913, of pneumonia, after an illness of less than one week. In his death the Engineering Profession has lost one of its great leaders; the community an active and efficient instrument for civic and social betterment; and his family and friends a wise counselor and a genial, lovable companion.

Mr. Adams was elected a member of the American Society of Civil Engineers on October 2d, 1895. From 1907 to 1909 he served as a Director. He was President of the San Francisco Association of Members of the American Society of Civil Engineers during 1907. He was a member of the Franklin Institute, the American Academy of Political and Social Science, the Technical Society of the Pacific Coast, the Pacific Association of Consulting Engineers, and of the First Presbyterian Church of Oakland. He held important offices in his church, and was Vice-President and Director of the Young Men's Christian Association of Oakland. He was also a member of the Delta Tau Delta and Sigma Xi college fraternities.

RICARDO MANUEL ARANGO, M. Am. Soc. C. E.*

DIED JANUARY 24TH, 1914.

Ricardo Manuel Arango, Consulting Engineer for the Republic of Panama, died at his home in the City of Panama, on January 24th, 1914, after a long illness. He was born on December 25th, 1864, and was graduated in 1887, with the degree of C. E., from the Rensselaer Polytechnic Institute, at Troy, N. Y. His first work after graduation was a survey, under the direction of the late Pedro J. Sosa, M. Am. Soc. C. E., a prominent engineer of Panama, of 500 000 hectares of land in the Province of Bocas del Toro, conceded by the Colombian Government to the French Canal Company. This party did the first surveying work in that region.

In 1889, Mr. Arango was appointed by the Colombian Government to take charge of surveys made necessary by a controversy between it and the Panama Railroad, over the filling of the present site of Colon. In 1890, he was engaged on the railroad which was built to haul to the coast the products of the manganese mines of Viento Frio, Province of Colon. He assisted in the surveys and plans for providing the City of Panama with water from the Juan Diaz River, but this development was never realized.

Mr. Arango took an active part in the Revolution of 1903 by which Panama secured its independence from Colombia, being of great assistance to his father, José A. Arango, who was a member of the original Junta of Separation. As member of the Municipal Council of the City of Panama, Mr. Arango signed the Act of Independence, and was appointed the first Chief Engineer of the new Republic.

Under John F. Wallace, Past-President, Am. Soc. C. E., Chief Engineer of the Isthmian Canal Commission, Mr. Arango was Consulting Engineer for sanitary work in the City of Panama. Later, he was made Division Engineer of the Bureau of Meteorology and River Hydraulics. As head of this Bureau, he installed the first seismograph on the Isthmus, and established the gauging station at Alhajuela, on the Chagres River, from which advices of any floods could be telophoned in advance to those in charge of work on the lower reaches of the river. His services with the Isthmian Canal Commission were terminated in the fall of 1908. Soon afterward Mr. Arango was appointed Minister Plenipotentiary and Envoy Extraordinary for the Republic of Panama to the Court of St. James, which post he was compelled to relinquish in 1909 on account of ill health. In 1910, he was again made Chief Engineer of the Republic, and, later, Consulting Engineer, which position he held at the time of his death.

* Memoir prepared by Alex. P. Crary, Assoc. M. Am. Soc. C. E.

He was a Member of the Instituto de Ingenieros de Chile, the Seismological Society of America, and the Sociedad de Ingenieros, Arquitectos y Agrimensores de Panama, of which latter he was the founder. On September 21st, 1899, he was married to Miss Maria Lewis, who, with five children, survives him.

Mr. Arango was a man who loved his profession. Even during his long illness, when he had lost almost completely the use of his hands, he took great delight in reading and studying engineering. In a wonderful manner he had trained his mind so that he could transform equations, construct graphical diagrams, and do things mentally which are generally accomplished by paper and pencil. He was always genial and willing to help and give advice. He stood up courageously under his sickness though it deprived him of the many pleasures and much work which make life worth living. He was always interested in public affairs, and, as long as he could, he took an active part in them. In his death, Panama loses one of its best and most progressive citizens.

Mr. Arango was elected an Associate Member of the American Society of Civil Engineers on September 2d, 1896, and a Member on February 6th, 1906.

HOWARD ELMER ARTHUR, M. Am. Soc. C. E.*

Died April 19th, 1914.

Howard Elmer Arthur was born at Warren, Ohio, on January 26th, 1872. He was educated in the public schools of Washington, D. C., and studied architecture for one year in the office of Messrs. Smithmeyer and Pelz of that city.

In August, 1891, Mr. Arthur came to New York City and entered the office of the Jackson Architectural Iron Works, where, for 5 years, he was engaged in detailing and checking the steelwork for office buildings, etc., erected by that firm. In August, 1896, he went to Paterson, N. J., where he was employed by the Passaic Rolling Mill Company, detailing and checking trusses, plate girders, etc.

In April, 1897, Mr. Arthur returned to New York City, where he was engaged, until December, 1898, with the firms of Cooper and Wigand and Lewinson and Just, in detailing and checking, and in charge of construction work. He resigned his position with the latter firm to go with the Union Bridge Company, at Athens, Pa., on detailing and checking and in charge of work. Slackness of work at the Union Bridge Company's plant caused Mr. Arthur to return to his former position with Messrs. Lewinson and Just in September, 1899, where he remained until March, 1900, when he went to St. Louis, Mo., with the Koken Iron Works, as Assistant Engineer. While in this position, he had charge of the construction of the Frisco Building, the Mississippi State Capitol Building, mining and mill buildings, tanks, etc. In 1902, Mr. Arthur was transferred to the Contracting Department of the American Bridge Company as Contracting Agent, and was also engaged on designing and estimating the cost of steel structures.

In September, 1904, he went to Dallas, Tex., as Chief Engineer of the Mosher Manufacturing Company, in which position he designed and estimated the costs for steel construction for various shops and buildings at Dallas and Houston, churches, highway bridges, tanks, domes, etc., as well as the Grand Stand at the State Fair Grounds at Dallas, which latter has been described by Mr. Arthur.†

In October, 1911, in order that his child, who was seriously ill, might receive proper medical treatment, Mr. Arthur returned to New York City. He entered the office of Olaf Hoff, M. Am. Soc. C. E., and, under Mr. Hoff's direction, made the drawings and calculations for the City Art Bridge from Fortieth to Forty-second Streets, connecting to the Grand Central Station, and various other constructions. He

* Memoir prepared by the Secretary from information on file at the Society House.

† *Engineering News*, Vol. LX, 1908, p. 208.

remained with Mr. Hoff until March, 1912, when, owing to failing health, he was compelled to resign. He spent the next two years in Vermont, Tryon, N. C., and the Catskill Mountains, in a vain endeavor to improve his health, but died of tuberculosis at Big Hollow, N. Y., on April 19th, 1914.

Mr. Arthur was regarded by all who knew him as a young man of high ideals, and his influence for good was felt by all who came in contact with him. The results which he accomplished bespeak his love for and devotion to his work.

On March 17th, 1905, Mr. Arthur was married to Miss Nettie Thomas, who survives him.

Mr. Arthur was elected a Member of the American Society of Civil Engineers on November 8th, 1909.

ADOLPHUS BONZANO, M. Am. Soc. C. E.*

Died May 5th, 1913.

Adolphus Bonzano was born at Ehingen, Wurtemburg, Germany, on December 5th, 1830. He was the youngest of four brothers who came to the United States, three of whom had distinguished careers. He, however, was the only one to follow the Engineering Profession.

Mr. Bonzano was educated at the gymnasia of Ehingen, Binsdorf, and Stuttgart. After the usual thorough training in these German schools, he came to Philadelphia, Pa., for further study, and particularly to perfect himself in the English language and in the customs of his adopted country. His father and other members of his family had emigrated to Texas in the Thirties, where they formed a number of German colonies in the interior, Gillespie and the adjoining counties, then an unsettled wilderness, but now part of the most prosperous portion of the State.

In 1852, Mr. Bonzano, who had early shown marked mechanical and engineering talents, recognized the great possibilities which the iron industry offered in the development of the country, and determined to supplement his academic studies by actual shop experience. He entered the Reynolds Machine Works, at Springfield, Mass., as an apprentice, and, at the end of his apprenticeship, became its Superintendent. For several years after this he was employed by several industrial and railway companies in various capacities in shop work, becoming one of the skilled mechanical superintendents of those days. During this period he became interested in bridge work and gave it such study as the early days of bridge history in the United States permitted, becoming a pioneer in its development. Finally, in 1865, he engaged with the Detroit Bridge and Iron Works as Superintendent of Bridge Construction, and, from that day until his retirement from business in 1898, he was an influential factor in the bridge industry, and in its formative period, particularly, his unusual talents were shown in his boldness of design, his advances in specifications, and his ingenuity in erection problems.

After serving three years with the Detroit Bridge and Iron Works, where his specialized work attracted much attention and earned him a broad acquaintance, he removed to Phœnixville, Pa., and with David Reeves, M. Am. Soc. C. E., and the late Thomas Curtis Clarke, Past-President, Am. Soc. C. E., assisted in the organization of the firm of Clarke, Reeves and Company (Phœnixville Bridge Company),

* Memoir prepared by F. A. Molitor, M. Am. Soc. C. E.

becoming a partner and its Chief Engineer. In the early days of the partnership, Mr. Bonzano made the plans, strain sheets, estimates, bids, and shop drawings. The firm rapidly became a leader and acquired the highest rank among bridge companies, many of the largest and most famous structures in the United States having been designed, built, and erected by it. In 1884, the firm was dissolved, being succeeded by the Phœnix Bridge Company, with Mr. Bonzano as Chief Engineer and Vice-President. For the next ten years he devoted all his time, experience, and business acumen to this company, his reputation probably reaching its zenith during this period.

The strain resulting from his professional activities of more than forty years, and the ever-increasing responsibilities of modern bridge business, determined him to reduce his activities, so he resigned from the Phœnix Bridge Company in 1893 and opened an office as Consulting Engineer in New York City, with his old friend and associate, the late Mr. Clarke, as partner. This association continued until 1898, when Mr. Bonzano retired from all active professional and business work, making his home in Philadelphia, Pa., where, surrounded by his family and many friends, he passed the evening of his life in quiet enjoyment.

In the work of the pioneer and formative period of American bridge construction, Mr. Bonzano had no peer. His unusual talents and attractive personality enabled him to place what was then bold and original bridge design under contract, and to this day many of his bridges are in use, carrying loads far in excess of their original design. He was also able to put the bridge business on a sound and proper basis, from specifications to erection, and in so doing he earned the appreciation of his associates and the grateful remembrance of their successors.

Mr. Bonzano's professional activities resulted in the building of many important bridges by his companies throughout America and in foreign countries, but only a few of these monuments to his genius will be mentioned here:

The Pecos Viaduct, 2 100 ft. long, carrying the Southern Pacific Railroad over the Pecos River at a height of 320 ft., built in 1890. The Red Rock Cantilever Bridge, over the Colorado River Canyon, on the Atlantic and Pacific Railway (now the Atchison, Topeka and Santa Fé Railway). The Kinzua Viaduct, 300 ft. high, on the Erie Railroad. At the time of its construction this viaduct was the highest structure in the world, and for boldness of design and erection methods astonished railroad and engineering circles, and was thoroughly discussed in the technical and daily press. The Chesapeake and Ohio Railroad Bridge carrying a double-track railroad, two roadways, and

two sidewalks, over the Ohio River at Cincinnati, built in 1888. This bridge, having one 550-ft. and two 240-ft. spans, was at that time the longest double-track span ever constructed. The Susquehanna River Bridge at Sunbury, Pa., built in 1882 for the Philadelphia and Reading Railroad, and the Columbia Bridge, Fairmount Park, Philadelphia, with seven spans aggregating 1 000 ft., built in 1886, also for the Philadelphia and Reading's double-track line. The Girard Avenue Bridge, at Philadelphia, 1 000 ft. long and 100 ft. wide, built in 1874, was one of the best examples of American municipal bridges.

Mr. Bonzano also had a large, if not the principal, share in the development of the modern draw-span, having designed and built some of the most notable structures of this type. The 274-ft. double-track draw of the New York Central and Hudson River Railroad, at Albany, built in 1870, was one of the first large railroad draw-spans. The construction of its turn-table embodied many original features designed by him, which later became standard practice. Some of the other and earlier draws built by Mr. Bonzano were the Harlem River Bridge, with a 300-ft. double-track draw-span, built in 1880; and the Albany and Greenbush, with a 400-ft. draw, carrying a double-track railway and roadway, built in 1881. As early as 1878 he built the 410-ft. through single-track draw-span of the Susquehanna Bridge, on the Philadelphia, Wilmington, and Baltimore Railway, then the longest draw in the world. All these bridges, except the Albany and Susquehanna draw-spans are still standing and carrying modern loadings.

Mr. Bonzano's inventive and mechanical genius was shown in the minor draw mechanisms. He was the first to use the locking roller with a pair of links at the draw end, and soon after modified this by the knuckle-joint. The use of a vertical screw for operating the locking mechanism was original with him.

He was also the pioneer engineer in the development of the first elevated urban railroads. The Sixth and Ninth Avenue Elevated Railroads in New York City with their Phœnix columns, now nearly 40 years old, are monuments to his boldness of professional thought as well as business judgment. He also built all the Kings County (Brooklyn Rapid Transit Company) Elevated Railroad lines.

The merits of the well-known Phœnix column, which was invented by the late Samuel J. Reeves, M. Am. Soc. C. E., of the Phœnix Iron Company, and was then only used in building construction, was at once recognized by Mr. Bonzano, and he was the first to introduce it in bridge compression members and to exploit its advantages. He made the designs of the details necessary to apply the Phœnix column to bridge construction, and it remained as the best bridge compression member for many years. This is shown by the fact that there

never was a failure of a Phœnix column in a bridge, no matter how much the latter was overloaded.

Mr. Bonzano gave many inventions to the modern world, as the records of the Patent Office testify. Chief of these is the popular rail joint, bearing his name, which he called his "little bridge", an epigrammatic expression which, with the ingenious thought that prompted the invention, was typical of him.

Although Mr. Bonzano will be remembered by the Profession as one of the pioneer and able bridge engineers of his age, his contemporaries will particularly remember him for his fine and unusual personality, and his lasting friendship for them. As an employer he was kindly and considerate, and many engineers of this generation will remember him for his helping and guiding hand when they were his apprentices and students. He numbered among his friends every one who knew him, his kindly disposition and genial manner making him a friend of everybody. It has been truly said that he never had an enemy. His charity, however, unostentatious and liberal as it was, will never be known; all we know is that he never refused a helping hand.

Mr. Bonzano was a Member of the Canadian Society of Civil Engineers, the American Society of Mechanical Engineers, the American Society of Mining Engineers, the Franklin Institute, the Union League Club of Philadelphia, and the Engineers Club of New York.

In 1857 he was married to Laura J. Goodell, in Detroit, Mich., and they had two sons, Hubert A. and Maximilian F. He is survived only by the latter, who is a Member of the American Society of Civil Engineers.

Mr. Bonzano was a talented musician, having been a skillful pianist and an able organist, and to the end he kept up a lively interest in all things musical. He was a familiar figure at the opera, and in his home life he always had much music. It was pleasant to be with him there, observe his love for music, and enjoy his unique and charming personality. His legion of friends and associates can do no more than cherish his memory and be glad that his long and busy life ended in a peace as beautiful as his beloved music.

Mr. Bonzano was elected a Member of the American Society of Civil Engineers on August 7th, 1872.

JAMES EDMUND CHILDS, M. Am. Soc. C. E.*

DIED JULY 16TH, 1912.

James Edmund Childs was born at Neversink, N. Y., on July 11th, 1848, and began his railway work, in April, 1865, with the Engineering Corps engaged in the location and construction of the New York and Oswego Midland Railroad, now a part of the New York, Ontario and Western Railway.

During 1869 and 1870, he was Assistant Engineer on the Chicago and Michigan Lake Shore Railroad, now a part of the Pere Marquette Line. Later in 1870 he was Resident Engineer of the Buffalo, New York and Philadelphia Railroad, now a part of the Pennsylvania System, and in the following year, 1871, Division Engineer of the Rochester and State Line Railroad, now a part of the Buffalo, Rochester and Pittsburgh Railroad.

In 1873 Mr. Childs was engaged in the relocation of a division of the Wisconsin Central Railroad, as Division Engineer, and, in 1874 and 1875, as Assistant Engineer in charge of some improvements on the New York and Harlem Railroad, now a part of the New York Central System.

In 1876, Mr. Childs returned to the Rochester and State Line Railroad as Chief Engineer and Superintendent, and five years later, in 1881, came back, as General Superintendent, to the re-organized New York and Oswego Midland, on which as a boy, sixteen years before, he had begun his railway work.

He was also Assistant General Superintendent of the New York, West Shore and Buffalo Railroad during its construction and at the time it was opened for traffic in 1883.

In February, 1886, he was made General Manager and, in September, 1904, Vice-President and Director, of the New York, Ontario and Western. Mr. Childs was in the service of this company continuously for more than thirty years, with the exception of one year, 1889, when he was offered and accepted the position of Assistant General Manager of the Lake Shore and Michigan Southern Railway properties.

In the following year, however, he returned to the service of the New York, Ontario and Western, at the earnest request of its President and Board of Directors, in order to forward and complete the Ontario, Carbondale and Scranton Railway, then under construction from the main line to the City of Scranton, through the upper anthracite coal region. Mr. Childs remained in the service of this company as General Manager until the time of his death, filling also the positions

* Memoir prepared from notes by William A. Haven, M. Am. Soc. C. E., by Edward Canfield, M. Am. Soc. C. E.

of Vice-President and Director. By his energy and devotion to its interests and his knowledge of the property and territory which it served, he did more than any one else to promote its welfare and increase its traffic. His long service with the officers and employees, and the intimate and friendly character of his relations with them, rendered his death not only a loss to the company, but a personal sorrow to his associates.

In 1882, Mr. Childs was married to Laura Grant, a daughter of the late William H. Grant, M. Am. Soc. C. E., who survives him.

Mr. Childs was elected a Member of the American Society of Civil Engineers on December 4th, 1878.

JOSEPH POTTER COTTON, M. Am. Soc. C. E.*

DIED DECEMBER 13TH, 1913.

Joseph Potter Cotton, a son of Isaac H. and Rhoda Lamont (Potter) Cotton, was born in Bowdoin, Me., on May 8th, 1837. He was descended from old New England stock. He received his education in the public schools and academies of his home district, and worked on the neighboring farms. Later, he taught in district schools in Maine, New Jersey, and Pennsylvania.

While he was engaged in teaching at Easton, Pa., the Civil War broke out, and, in 1862, he served as Orderly Sergeant of a company of militia at that place. In 1863, he raised and took command of Company C, Forty-eighth Regiment, Pennsylvania Volunteers, which was mustered into the service of the United States.

In 1866, Captain Cotton began his engineering work as Rodman and Leveler on the survey for the Lake Superior and Lake St. Croix Railroad in Wisconsin. In September, 1867, he entered the Government service, under the late General G. K. Warren, Corps of Engineers, U. S. A., and was engaged as Assistant Engineer on surveys of various rivers and bridges in Minnesota, Mississippi, Wisconsin, and Ohio. In 1871, he went to Newport, R. I., and, under General Warren, had charge of the construction of the breakwater at Block Island. In 1872 and 1873, with the late E. S. Chesbrough, Past-President, Am. Soc. C. E., he made a survey of Newport, in connection with a plan for a new system of sewerage, which plan, slightly modified, was adopted and forms the basis of the present system. Captain Cotton continued in the service of the Government until 1883, being engaged on river, harbor, and fortification work. In 1882 and 1883, he served as Commissioner and Engineer on the Seekonk River Bridge, at Providence, R. I.

In 1883, he resigned to engage in the private practice of engineering. He made his home at Newport, and became actively interested in city affairs and public improvements. From 1876 to 1883, he served as a Member of the School Committee; he was also Overseer of the Poor for three years, and Street Commissioner in 1890 and 1891. He was one of the group of citizens who framed the present city charter, and, on its adoption, he became a member of the Representative Council, serving on several of its important committees. He continued as a member of the Council until his death.

Captain Cotton was also deeply interested in the social betterment of the city and its people. He was one of the founders of the Charity Organization in 1880, and from that time until his death was an ac-

* Memoir prepared by the Secretary from information on file at the Society House.

tive member of its Board of Reference. He assisted in establishing the Holly Tree Coffee Rooms, the Law and Order League, and the Industrial School, and was for some years a Trustee of the Newport Hospital. His greatest public service, and one in which he took the greatest interest and pride, was rendered in connection with the Newport Co-operative Society for Saving and Building, which he was largely instrumental in founding in 1888, and of which he was the first and only President. The growth, prosperity, and efficiency of this institution is owing more to him than to any other man.

On March 26th 1867, he was married to Miss Isabella Cole, who died in 1908. He is survived by two sons, Dr. Frederic J. Cotton, of Boston, Mass., and Joseph P. Cotton, Jr., of New York City.

Captain Cotton was actively engaged in work up to the very end of his long life. He died in his sleep on December 13th, 1913, without pain and without premonition of death.

A few days later, Rear-Admiral French E. Chadwick, U. S. N., retired, wrote of him:

"Our town feels that in the death of Captain Cotton it has lost a mainstay. This loss is not only in our being deprived of a wise counsellor, an upright, clear-thinking citizen working always for the good of his fellow-men, but we have lost an exemplar of character. To old men he was an example, in age, of activity in business and public life; to the younger he was a pattern, for he exemplified in very full degree what character in the large sense means."

Joseph Potter Cotton was elected a Member of the American Society of Civil Engineers on June 7th, 1876.

FREDERIC DANFORTH, M. Am. Soc. C. E.*

Died June 6th, 1913.

Frederic Danforth, the son of Judge Charles and Julia S. (Dinsmore) Danforth, was born at Gardiner, Me., on February 8th, 1848. He was graduated from Dartmouth College (Thayer College of Civil Engineering) in 1870, and began work as Rodman on the construction of the European and North American Railroad, in August, 1870.

From that time until his death, he was almost constantly connected with railway work in various capacities, being employed as follows: by the Augusta and Wiscassett Railroad, from October, 1871, to May, 1872; Bartlett Division of the Portland and Ogdensburg Railroad (now the Mountain Division of the Maine Central Railroad), from August, 1872, to October, 1873; Boston and Northwestern Railroad in 1874; West Waterville and Augusta Railroad and the Augusta and Lewiston Railroad in 1875; the European and North American Railroad and the Penobscot River Railroad, from 1876 to 1881; the Gardiner and Farmington Railroad and the Aroostook Central Railroad in 1881; the Skowhegan and Athens Railroad in 1882; the Maine Shore Line Railroad and the Mt. Desert Railway in 1883; the Franklin and Megantic Railway in 1884; the Northern Maine Railroad, as Chief Engineer, in 1888 and 1889; the Kennebec Central Railroad, as Chief Engineer, from 1888 to 1890; the Portland and Rumford Falls Railroad, as Chief Engineer, from 1891 to 1894; the Maine Central Railroad in 1901; the Debec Junction Railroad in 1905; the Eastern Maine Railroad, as Chief Engineer, in 1911 and 1912.

Contemporaneously with this work, Mr. Danforth was engaged in land and city surveys in Gardiner, Me., and in surveys of townships in the State. In 1882, he designed the dam for the Augusta Water Company. He also served as City Engineer of Gardiner for a long period.

In November, 1894, he was appointed Engineer Member of the Board of Railroad Commissioners of Maine, and was re-appointed in November, 1897. He was elected Mayor of Gardiner in March, 1901. and re-elected in 1902; he also served as a member of the Board of Trustees of the Gardiner Water District in 1904. Mr. Danforth was frequently employed as Supreme Court Referee in the adjustment of disputed township boundaries and other engineering questions. On his appointment as member of the Board of Railroad Commissioners. he resigned his official connection with several railroads in Maine, and thereafter practiced extensively as Consulting Engineer, especially

* Memoir prepared by W. W. Crosby, M. Am. Soc. C. E.

for railroads and bridges. At the time of his death, he was a Director of the Gardiner National Bank.

In 1880 Mr. Danforth was married to Miss Caroline Stevens, of Randolph, Me., who, with four children, survives him.

Mr. Danforth was a man with great practical, as well as theoretical, knowledge, and of rare common sense. He was a careful, accurate, and hard worker, and, in railroad location, had few, if any, equals, his methods for such work embodying the highest kind of science. He had strong domestic tastes, and was kindly and considerate in the extreme, though of a retiring disposition. He inspired the sincere affection and regard of his subordinates and of all who knew him, and his judgment and advice were frequently sought and always generously given. He was highly respected everywhere for his character and ability. He was devoted to his profession and inspired that devotion in the other members of it with whom he came in contact. Truth and honesty stood above all to him, and in all his life he unswervingly insisted that every action or decision should be based on them. The Profession, as well as his own immediate circle, suffers an immeasurable loss in his death.

Mr. Danforth was a Member of the Thayer Society of Civil Engineers and the Maine Society of Civil Engineers. He was elected a Member of the American Society of Civil Engineers on September 2d, 1891.

JAMES JOSEPH FERRIS, M. Am. Soc. C. E.*

DIED MAY 15TH, 1914.

James Joseph Ferris, the son of John and Mary (Harrington) Ferris, was born in Listowel, County Kerry, Ireland, on September 17th, 1860. When he was five years of age, his family came to the United States and settled at Carmel, Putnam County, New York, where they lived for six years, until ill-health and business reverses caused their return to Ireland. Mr. Ferris' early education, started at Carmel, was completed in Ireland, and, after leaving school, he helped his father, who was the manager of a large estate.

After a few years at farm work and at the age of twenty, Mr. Ferris returned to the United States and settled in Jersey City, N. J. He entered the employ of D. S. Cofrode and Company, as Timekeeper, and, subsequently, as Foreman, on important bridge work and other heavy railroad construction in Maryland, Pennsylvania, and Virginia.

Early in 1886, during the settlement of the estate of this firm, he was made Superintendent by the executors, and was in charge of all the work being done by the firm. When a re-organization was effected in 1888, under the firm name of Sandford and Stillman Company, Mr. Ferris was appointed General Superintendent, and held this position until 1902, when the firm was re-organized and became known as The F. M. Stillman Company, Mr. Ferris remaining as General Superintendent. In 1911, the firm was again re-organized under the name of Stillman-Delehanty-Ferris Company, Engineers and Contractors, and Mr. Ferris became Vice-President and Chief Engineer, in charge of all engineering and outside work. About a year ago (1913), he became President of the Company.

Mr. Ferris' engineering education was obtained at Cooper Union Institute in New York City, and through constant home study. He was a keen observer, possessed a wonderful memory, and qualities for thoroughness, ready debate and argument, and was of a most genial and happy nature. Accustomed to directing the labor of large numbers of men, he had fine tact, most courteous manners, and, in a very high degree, the power to command loyal service and friendship from others.

He was in charge of various parts of the construction of the Pennsylvania Railroad Company's train-shed and office building, and the ferry slips and sheds at Jersey City, and directed the work of the grade crossing elimination for the same railroad through that place. He was also in charge of the construction of the roundhouse and yards of the New York, Susquehanna and Western Railroad, at Shady-

* Memoir prepared by Louis Chevalier, Assoc. M. Am. Soc. C. E.

side, and of various other work about New York Harbor, notably the sinking of the shafts at Washington Street and at Exchange Place, in Jersey City for the tube elevators for the Hudson and Manhattan Railroad Company, and the construction of the Hackensack River Turnpike Bridge.

In June, 1913, Jersey City adopted a Commission form of Government, and Mr. Ferris was signally honored by being elected a member of the first Board to govern its civic affairs. He was selected to take charge of the Department of Streets and Public Improvements, and during the past year he had brought his Department to a high state of efficiency, his most notable achievement being a thorough cleaning up of the City's water supply on the Boonton water-shed.

Mr. Ferris died suddenly on May 15th, 1914. He was a thorough and painstaking engineer, an exceptionally devoted student, was esteemed and admired by all who knew him, a faithful friend, a devoted father, and a good citizen.

In 1888, he was married to Miss Bridget Ryan, of Baltimore, Md., who, with five sons and three daughters, survives him.

Mr. Ferris was elected an Associate of the American Society of Civil Engineers on July 10th, 1907, and a Member on September 3d, 1912.

JOHN DOUGLAS FOUQUET, M. Am. Soc. C. E.*

Died September 18th, 1913.

John Douglas Fouquet was born at Plattsburg, N. Y., on August 1st, 1829. He received his early education from governesses and at the Parish School. When he was sixteen years old, he was enrolled at the Plattsburg Academy, where he received his preparatory training. He entered Rensselaer Polytechnic Institute in 1849, taking the course in Civil Engineering.

In 1852, at the end of the Spring term, Mr. Fouquet left Rensselaer to accept a position as Draftsman on stations and buildings with the Plattsburg and Caughnawaga Railroad, which later became the Plattsburg and Montreal Railroad, and is now a part of the Delaware and Hudson System. On May 1st, 1854, he became a member of the Engineer Corps of the Sunbury and Erie Railroad, now the Philadelphia and Erie Branch of the Pennsylvania Railroad, with headquarters at Williamsport, Pa. He was engaged first as Assistant to the Planetable Topographer on the preliminary and location surveys of the road and, on their completion, was retained, under Mr. Robert Ferris, Chief Engineer, and Col. Phaon Jarrett, Assistant Chief Engineer, as Head Draftsman on the Eastern Division of the road (Ridgway and Sunbury Railroad), on the preparation of plans for the stations, buildings, culverts, and seven large Howe truss bridges, each 1 000 ft. long, at various crossings of the West Branch of the Susquehanna River.

On account of the panic of 1857, work on the Sunbury and Erie Railroad was suspended for six months, and during this time Mr. Fouquet was engaged under the late P. P. Dickinson, M. Am. Soc. C. E., Chief Engineer, as Leveler on the preliminary and location surveys on a line of about 40 miles, between Dauphin and Landisburg, Pa., for the Sherman Valley and Broadtop Railroad, which road was projected in connection with the Broadtop Coal Mines.

When work on the Sunbury and Erie Railroad was resumed, he returned to that road, with headquarters at Lock Haven, Pa., remaining with the Company until the fall of 1860, when the line was acquired by the Pennsylvania Railroad, at which time Mr. Fouquet returned to Plattsburg, N. Y. During the summer of 1861 he was engaged by the State of New York to survey and map the State prison property at Dannemora, N. Y., and, in connection with this work, he designed a ventilating shaft for the iron mines within the enclosure of Clinton Prison.

On November 18th, 1862, Mr. Fouquet entered the United States Navy as Clerk to Rear-Admiral Theodorus Bailey, commanding the

* Memoir prepared by the Secretary from information on file at the Society House.

East Gulf Blockading Squadron, on the flagship *St. Lawrence,* which was stationed at Key West, Fla., part of his duties consisting in bearing dispatches to and from the U. S. Consul at Havana, Cuba.

In May, 1864, he resigned from the Navy and was appointed to a position at the Brooklyn Navy Yard by Gideon Welles, the Secretary of the Navy. On his arrival in New York City, however, he was offered and accepted a more lucrative position with the Athens and Schenectady Railroad as Topographer in the field organization then being formed by that road, with headquarters at Athens, N. Y. In this position he was engaged on preliminary and location surveys, and, later, on construction, in charge of the Terminal Division, which included docks, bulkheads, passenger stations, as well as engine-houses, shops, tenements, etc., at Athens.

In the fall of 1866, Mr. Fouquet entered the employ of the Atlantic and Great Western Railroad as Chief of Topography, on the staff of Col. James Worrell, who was Chief Engineer of the Sir Morton Peto project for an air-line from New York City to Chicago, with headquarters at Harrisburg, Pa. The work consisted in closing up gaps between existing lines in New York, Pennsylvania, and Ohio. There were five engineering corps in the field between Lewisburg, Pa., and Greenville, Ohio. In the spring of 1867, however, the failure of Sir Morton Peto caused all work to be abandoned, and the various corps were left stranded in the oil regions of Pennsylvania.

Mr. Fouquet then became Topographer, under the late Oliver W. Barnes, M. Am. Soc. C. E., Chief Engineer, on the preliminary and location surveys for the Dutchess and Columbia Railroad, later known as the Newburg, Dutchess, and Connecticut, and now the Central New England Railroad. He was afterward made Division Engineer on the construction of the Western Terminal Division, with headquarters at Fishkill, N. Y., and later became Architect for the terminal and station buildings.

On the completion of this work, Mr. Fouquet engaged in the private practice of civil engineering and architecture, with an office at Fishkill, N. Y. He designed the building for the Fishkill High School, and was engaged in engineering and architectural work by the Dart Manufacturing Company in connection with a large woolen mill plant at Glenham, N. Y. He was also engaged on work for the Whiting Junction Railroad of Vermont, now a part of the Vermont Central System, one interesting feature of this work being the construction and installation of a pontoon draw in connection with the maintenance of the channel in Lake Champlain.

In the spring of 1871, Mr. Fouquet retired from private practice to accept a position as Engineer and Architect with Garner and Company, of New York City, which Company owned large cotton mills and print works at various places in New York, Pennsylvania, and

Rhode Island. In this position he designed and superintended the construction of many mills, warehouses and tenements in New York State, as well as dams, reservoirs, and connections with existing railroad lines, together with the reconstruction and additions to various plants belonging to the Company. He also designed a Club House at Stapleton, S. I., and rebuilt a New York residence, for Mr. William T. Garner, the President of the Company.

While in this position, Mr. Fouquet designed the large warehouse at the corner of Worth and Hudson Streets, in New York City, which is still in use. At that time this building was second in height to the Western Union Building, then the tallest commercial building in the city.

In the fall of 1873, when the business portion of Fishkill, N. Y., was nearly wiped out by fire, Mr. Fouquet opened a branch office there, and designed stores, dwellings, bank buildings, etc. In 1882, he resigned his position with Garner and Company to accept an appointment as Chief of the Construction Department of the New York, West Shore and Buffalo Railway (later the West Shore Railway), with headquarters at Jersey City, N. J. The work included the design and construction of bulkheads, piers, transfer and ferry bridges in New York Harbor, etc., the offices being subsequently moved to New York City, and finally to Weehawken, N. J.

In the fall of 1883 the Company was placed in the hands of a Receiver. With a few others, Mr. Fouquet was retained and made Division Engineer of the Western Division, with headquarters at Frankfort, N. Y. He remained at that place until the completion of the extensive yards and shops in 1884, when he was transferred to Syracuse, N. Y., and placed in charge of the completion of more than 100 stations, freight-houses, etc., between Newburgh and Buffalo, N. Y. In the fall of 1884, he left Syracuse and made his headquarters at Weehawken, N. J. In 1885, work on the West Shore Railroad was entirely completed, and Mr. Fouquet returned to his home in Fishkill, N. Y. Within a week, however, he was recalled to prepare the plans and take charge of the construction of the Forty-second Street Terminal Station in New York City, which had been destroyed by fire. He also prepared the plans for the Company's terminal at Jay Street.

In 1886, the West Shore Railroad was leased by the New York Central and Hudson River Railroad, and Mr. Fouquet was transferred to the Grand Central Station and made Architect of the combined systems. In addition to his architectural work, Mr. Fouquet had charge of the surveys for a branch road from the West Shore Railroad at Saugerties, N. Y., to the Harding Hotel on the summit of the Catskill Mountains, a distance of about 9 miles, the maximum grade being about 80 per cent. He also had charge of all architectural and construction work in connection with the depression of the tracks of the

New York and Harlem Railroad between Mott Haven and Woodlawn Junction, the elevation of the tracks on the viaduct from the north end of the Fourth Avenue Tunnel to Mott Haven Junction, and the construction of the large storage yards and shops at Mott Haven.

On September 1st, 1893, Mr. Fouquet resigned his position with the New York Central and Hudson River Railroad Company to engage in private practice in New York City as Consulting Engineer and Architect. In this capacity he made the plans and estimates for the elimination of grade crossings for the City of New Bedford, Mass., designed the Fireboat Station at Battery Park, for the City of New York, and was associated with the R. P. and J. H. Staats Company in the design of the façades for the bulkheads and piers for the Cunard, White Star, and Wilson-Furness Steamship Lines, on West Street, New York City.

In 1900, Mr. Fouquet was obliged to give up his work on account of serious illness. On his recovery he again entered the employ of the New York Central and Hudson River Railroad, but in January, 1905, he was so seriously injured by an accident in one of the passenger elevators at the Grand Central Terminal that he retired from active work and returned to Fishkill, where he resided until his death.

Mr. Fouquet was engaged actively in the practice of his profession for a period of 53 years, and the fact that during that time he had had charge of millions of dollars worth of work, and had never been long without employment, was always a source of gratification to him. He had sustained at various times fractures of both arms, and through the improper bandaging of one such fracture, had since the age of twelve years been confined to the use of his left hand.

In his early youth he had developed artistic talent, and at odd times and for his own pleasure, he worked in both oils and water colors. His leisure was devoted to hunting and fishing, from which recreations he derived much pleasure.

On December 27th, 1864, Mr. Fouquet was married, at Athens, N. Y., to Miss Emma J. Leffingwell. He is survived by two sons, Louis Douglas Fouquet, Engineer of the Sewer Division of the Public Service Commission of the First District, State of New York, and Morton Leffingwell Fouquet, Engineer in Charge of the Department of Substructures of the Borough of Brooklyn, New York City.

He was a Member of U. S. Grant Post, G. A. R., No. 327, of Brooklyn, N. Y. From his youth he had been a member of the Protestant Episcopal Church, and had always taken an active interest in church work, having served for many years as Vestryman at St. Stephen's Church, in New York City, and as Vestryman and Junior Warden of Old Trinity, in Fishkill, N. Y.

Mr. Fouquet was elected a Member of the American Society of Civil Engineers on June 3d, 1885.

WEBSTER GAZLAY, M. Am. Soc. C. E.*

DIED FEBRUARY 17TH, 1914.

Webster Gazlay was born in Louisville, Ky., on February 17th, 1862, and given the name of "Webster" in recognition of his father's admiration for the great statesman. His surname has come through mutations from Gazeley, Gazelle, Geasley, and Gaselee, of early England. For the past 400 years, Judges of the name of Gaselee have sat on the King's Bench, in England, and their decisions are still quoted in English books of law. A Judge Gaselee was Dickens' prototype, it is said, for "Judge Stareleigh" in Pickwick's famous breach of promise suit.

Mr. Gazlay's mother was a Wheeler, of Virginia ancestry, Chaplines and Clairbornes blending. On her maternal side, John Randolph, of Roanoke, was a kinsman.

In 1715 (Old Style) the English progenitor of the family in America settled in the State of New York, and his son, John Gazlay, was a member of the Continental Congress. Webster Gazlay's father, Addison M. Gazlay, was born in the valley of the Unadilla River, in Central New York, went to Kentucky in his early manhood, and for many years was a leading commercial lawyer at the Louisville bar.

In spite of the marked inclination in the family to the pursuit of law, Mr. Gazlay early manifested such a love and aptitude for mathematics, that he easily stood at the head of his classes in such studies. This passion for mathematical investigation, undoubtedly influenced his choice of a profession and determined the precision of thought and act and the exactness in expression and language, that were matters of comment and banter among his intimate friends.

He was graduated from the Louisville Male High School in 1881, and, the death of his father preventing the further prosecution of his technical studies, he accepted a position as Draftsman in the office of the Chief Engineer of the Louisville and Nashville Railroad, where he remained until May, 1883.

After leaving the Louisville and Nashville Railroad Company, he was appointed Assistant to the late Charles Hermany, Past-President, Am. Soc. C. E., and was engaged on the construction of water-works at Frankfort, Ky., and on surveys, plans, etc., for a proposed reservoir for Nashville, Tenn.

On the completion of these projects, he became Assistant Engineer on the Louisville Southern Railroad (now a part of the Southern Railway System), and was in charge of a residency during the construction of that road.

* Memoir prepared by J. M. Johnson, M. Am. Soc. C. E.

In 1888, Mr. Gazlay accepted a position, as Assistant Engineer, with the Louisville Water Company, later becoming Assistant to the Chief Engineer, during the construction of a new pumping station and an extensive system of filters; he remained in this position until 1905, resigning at that time to become the Vice-President and Engineer of the National Concrete Construction Company, a contracting firm with its principal office at Louisville, Ky.

In 1906, he again returned to the Louisville Water Company as Associate Engineer, and retained this position until the completion of the filter plant in the Fall of 1909. In accepting his resignation, the Water Board passed the following resolution to be recorded in its Minutes:

"To his intelligence in conception, vigor in execution, and tireless energy in the carrying on of the work, is due, in large measure, the successful completion of the great enterprise to which he has devoted himself."

Mr. Gazlay was married on November 7th, 1906, to Miss Lida Hampton, of Louisville, Ky., who, with three children, Webster Gazlay, Jr., Lida Hampton Gazlay, and Sallie Josephine Gazlay, survives him.

Mr. Gazlay was a man of high ideals and thoroughly in love with his Profession. All his concepts were strong and clear, touched in a great degree with that light and leading which humanity is wont to term genius. His standard condemned anything less than the wholly clean way of honesty and truth, and permitted no compromise where the facts were clear and plain. To his many friends, his untimely passing away is a source of great sorrow, but they have the consolation of knowing that it has been their privilege to work and walk with a man fashioned well and truly in the image of his Maker.

He was one of the original members of the Engineers and Architects Club of Louisville, Ky., and was always placed on committees where important and efficient work was demanded.

Mr. Gazlay was elected a Member of the American Society of Civil Engineers on June 7th, 1905.

CHARLES ARTHUR HAGUE, M. Am. Soc. C. E.*

Died June 26th, 1911.

Charles Arthur Hague was born at Newton, Mass., on October 9th, 1849.

He was in the employ of the Clapp and Jones Manufacturing Company at Hudson, N. Y., as a Draftsman and Designer, from 1872 to 1875, when he resigned to become Mechanical Engineer and Draftsman on steam engines and boilers for the Frank Douglas Machinery Company, of Chicago.

In 1876, Mr. Hague became Master Mechanic for the Furst and Bradley Manufacturing Company, and while in that position he patented many improvements on plows and other implements, besides designing and erecting numerous special machines. From 1884 to 1887, he was Superintendent for the E. P. Allis Company, of Milwaukee, Wis.

After one year with the Knowles Steam Pump Company, of New York City, he became Mechanical Engineer for the H. R. Worthington Company, which position he held until 1895. From that time until his death, he was engaged in consulting practice in New York City.

Mr. Hague was the author of a treatise entitled "Pumping Engines for Water Works." He was a member of the American Society of Mechanical Engineers, the American Water Works Association, and the New England Water Works Association.

Mr. Hague was elected a Member of the American Society of Civil Engineers on February 3d, 1892.

* Memoir prepared by George A. Orrok, M. Am. Soc. C. E.

JAMES CHARLES HAUGH, M. Am. Soc. C. E.*

Died July 6th, 1913.

James Charles Haugh, the son of Thomas and Jane Watts Haugh, was born in Cincinnati, Ohio, on March 23d, 1855.

His primary education was received in the public schools of Cincinnati. At an early age he entered the office of Gen. A. J. Hickentopper, then City Surveyor, but pursued his studies at the High School by a night course. After some years in the City Surveyor's office, during which he had earned promotion, he entered the service of the Portsmouth and Ohio Railroad, from which employment he shortly resigned to become Resident Engineer of the Cincinnati Southern Railroad, in charge of tunnel arching and grade construction. In this work he was associated with the late Col. George B. Nicholson, M. Am. Soc. C. E., who had been the friend of Mr. Haugh's brother. The association of Mr. Haugh and Col. Nicholson was intimate until the death of the latter.

In 1881 Mr. Haugh was sent by Col. Nicholson to the New Orleans and North Eastern Railroad and had charge, as Resident Engineer, of the construction of the Lake Pontchartrain Trestle (21½ miles long) which at that time was the longest trestle in the world. On the completion of the construction of the New Orleans and North Eastern Railroad, Mr. Haugh was placed in charge of Maintenance of Track, Bridges, and Buildings, in which capacity he served until his death.

Mr. Haugh was a Member and Past-President of the Louisiana Engineering Society, and also a Member of the American Railway Engineering Association.

His information on all matters pertaining to the maintenance of railways was thorough, and he was perhaps one of the best informed engineers in the United States on timber preservation.

Mr. Haugh had a unique personality. He possessed a shyness which bordered almost on diffidence, yet a latent force pervaded his every thought and undertaking. He was exceptionally charitable, his charity being of the unostentatious kind. His unfailing cheerfulness was in itself an inspiration to those with whom he came in contact.

It is difficult to express in words the affection in which Mr. Haugh was held by his associates. His circle of acquaintances was large, and every member of it feels a sense of personal loss in his death. Those who were privileged to know him well believe that the world is better by reason of his sojourn therein.

Mr. Haugh was elected a Member of the American Society of Civil Engineers on February 2d, 1909.

* Memoir prepared by W. B. Gregory and J. F. Coleman, Members, Am. Soc. C. E.

JOHN WILLIS HAYS, M. Am. Soc. C. E.*

DIED DECEMBER 14TH, 1913.

John Willis Hays was born at Oxford, N. C., on March 14th, 1861, and was christened with the name borne by his father and his grandfather. He was educated at local private schools and the University of North Carolina.

As a young man Mr. Hays did engineering work under the late Benjamin D. Frost, M. Am. Soc. C. E., of Massachusetts, who had distinguished himself in connection with the building of the Hoosac Tunnel in that State, and was, at the time in question, engaged in laying out a route for the Oxford and Henderson Railroad. Later, Mr. Hays joined the field staff of William C. Kerr, State Geologist of North Carolina, where he acquired experience and developed ability which gained for him an appointment as a Topographer in the United States Geological Survey. This position he held for many years, first in the Appalachian Mountains, and later in the Rockies, during the summer, and at Washington in winter.

In addition to the remarkable thoroughness of his engineering work, Mr. Hays was distinguished for the unusual excellence of his drafting. Indeed, his artistic ability was marked, and had he chosen Art as a vocation, his success would doubtless have been gratifying. Nor was his facility with the pen confined to drafting and free-hand sketching, for many of his exciting mountain adventures were reduced to manuscript and published in such periodicals as *The Outing Magazine, The Youth's Companion,* and the newspapers of Washington and other cities. He was also an occasional contributor to journals devoted to engineering matters.

Leaving the Government service in the early Nineties for work in which his individuality would have freer play, Mr. Hays became City Engineer of Petersburg, Va., having a few years previously married the daughter of one of the leading citizens of that place, who, with a large family, survives him. While in that position he laid out Ferndale Park, constructed the high-service reservoir, and in other ways left his impress on the city's physical characteristics.

Mr. Hays' nature was such that he could not be contented in a political position. He knew of but one course to pursue, and that was the one of absolute rectitude as he saw it, and he was remarkably free from moral strabismus. Consequently, he left the public position to engage in private work. Although frequently urged to return to the City Engineership at an increased salary, he steadfastly and politely declined.

* Memoir prepared by Francis B. Hays, Esq.

In the meantime, he built up a private practice which became more remunerative as the years went by, and he was able to command the fee of an expert for a mere opinion. Shortly before his death, he had completed the development of the Walnut Hills Section, reached from the City of Petersburg proper by an iron viaduct over a broad gorge which latter had long rendered it practically inaccessible. He developed the water power at Roanoke Rapids, N. C., and did municipal work at Mt. Airy, Dunn, Kinston, and other towns in his native State, as well as at Emporia, Blackstone, and many other points in the State of his adoption.

Physically, Mr. Hays was a man of magnificent mould, having been considerably more than 6 ft. in height, erect and clean-limbed, and carrying his two hundred and odd pounds without the suggestion of a surplus ounce of flesh. He was cut down at the zenith of his powers by thrombosis induced by a fall sustained while engaged in field work.

Mr. Hays was Master of Blandford Lodge of Masons, President of the Petersburg Benevolent Mechanics' Association, an Elk, and, for several years, had conducted with marked success a men's Sunday-school class. He was a man of vigorous mind, keen intellect, catholic tastes, strong personality, a faithful friend, and a true gentleman.

Mr. Hays was elected a Member of the American Society of Civil Engineers on June 5th, 1901.

WILHELM HILDENBRAND, M. Am. Soc. C. E.*

DIED FEBRUARY 21ST, 1908.

Wilhelm Hildenbrand, one of the most eminent designers and constructors of suspension bridges, was born on June 1st, 1843, at Karlsruhe, in the Grand Duchy of Baden, Germany. After receiving a classical education in the Lyceum of his native town, he entered its Polytechnic School, which was then the most renowned in Germany, having among its professors, Redtenbacher, one of the founders of scientific mechanical engineering, and Sternberg, equally prominent in scientific bridge design. After he was graduated in engineering and had passed the "Staats Examen" required for Government employment, he entered its service as Engineer of Highway Construction and as Inspector of rails at the rolling mills of Westphalia and Rhenish Prussia. After a year in the service of the State, he emigrated in 1867 to America.

His first position was as Draftsman in an architect's office, but he soon found employment with the late John A. Roebling, M. Am. Soc. C. E., who was at that time planning the construction of the Brooklyn Bridge. Under Mr. Roebling's directions, Mr. Hildenbrand made the first drawings of that famous structure, in the building of which he afterward took a prominent part.

During the delay caused by the initial difficulties of this enterprise he entered, as Assistant Engineer, the service of the New York Central Railroad where he was entrusted with the architectural design for the Forty-second Street Terminal Passenger Station in New York City, and with the construction of the great arched roof over the train-shed. Both structures were built in accordance with his plans and specifications, thus laying the foundation of his reputation as an architect and engineer. The roof truss had the largest span in the world at that time, and lasted until the entire reconstruction of the station required its removal.

In 1870, when the Grand Central Station was nearly completed, the actual construction of the Brooklyn Bridge began, and Mr. Hildenbrand was engaged by Washington A. Roebling, the Chief Engineer, as Principal Assistant Engineer. During Mr. Roebling's protracted illness, which confined him to his bed or room for nearly ten years, Mr. Hildenbrand made all the scientific investigations and mathematical calculations necessary for the structure. He also made the architectural design for the approaches and had charge of the steel superstructure, which was designed, inspected, and erected under his direction.

In 1883, after the completion of the Brooklyn Bridge, Mr. Hildenbrand opened an office in New York as Consulting Engineer and, as

* Memoir prepared by Joseph Mayer, M. Am. Soc. C. E.

Chief Engineer, built many suspension bridges in the United States and Mexico. He also built a truss bridge over the Ohio River at Wheeling, W. Va., which, like most of his work, is distinguished for its pleasing appearance.

In 1885 he submitted a design in the public competition for the Washington Bridge, and was awarded the second prize. Several features of his design were incorporated in this bridge as built.

In 1894 Mr. Hildenbrand did important work for the New York Chamber of Commerce which protested against the building of a pier near the center of the North River for a railroad bridge at that time projected by the New York and New Jersey Bridge Company. The President of the United States appointed a commission of five distinguished engineers to examine the feasibility of a single-span bridge across the river. Mr. Hildenbrand submitted a design, an estimate, and an argument in favor of a suspension bridge, and appeared before the Commission as the representative of the Chamber of Commerce. The Commission reported in favor of the feasibility of a suspension and against a cantilever bridge, which was the question at issue; Mr. Hildenbrand, therefore, carried his point.

In 1895 he became Chief Engineer of the reconstruction of the Covington and Cincinnati Bridge over the Ohio River. This was his most important and difficult independent work. The bridge is the third largest suspension bridge in the world, and was built originally by Mr. John A. Roebling in 1867, but had become inadequate for the increased traffic. Mr. Hildenbrand replaced the floor and the stiffening trusses with a wider floor and new stiffening trusses without interrupting traffic, and supplemented the old cables and anchorages by additional ones. The task of distributing the load between the new and the old cables was difficult, and required accurate calculation and delicate adjustment. It was evidently accomplished successfully; the new bridge is not only an adequate but a beautiful structure, and an ornament to the two cities. It was finished in 1899, to the satisfaction of his clients.

On the completion of this bridge, Mr. Hildenbrand went to Mexico, where he constructed a light suspension bridge, of about the same span as the Cincinnati and Covington Bridge, for the transport of silver ore from the mines of the Peñoles Company at Mapimi to the terminal point of a rack railway, built on the Abt system, which Mr. Hildenbrand had constructed in 1897.

As the representative of the Abt system of inclined rack railways, Mr. Hildenbrand, in 1889-1890, constructed the track of the Pike's Peak Railway in Colorado, reaching an elevation of 14 214 ft., and furnished its locomotives.

In 1900, in the service of John A. Roebling's Sons Company, of Trenton, N. J., he became the Chief Engineer of the contractor for building the cables of the Williamsburgh Bridge. These cables, 50%

larger than those of the Brooklyn Bridge, were built by him in one-third of the time required to construct those of the Brooklyn Bridge.

In 1904, he was appointed Consulting Engineer of the Westinghouse Electric Company, of Pittsburgh, and as such he designed the overhead structures for the electrification of the passenger traffic of the New York, New Haven and Hartford Railway, near New York City. He designed other and similar structures for the Westinghouse Company, acting as Consulting Engineer for this Company until his death.

In 1877 Mr. Hildenbrand published a treatise on "The Theory and Construction of Wire Cables," which is a standard work on the subject. In 1888 he published a book on "Underground Haulage of Coal by Wire Rope," which was awarded a medal and diploma of honor at the World's Fair of Chicago in 1893.

In 1895 he published in the German language a lecture on the history of suspension bridges from the earliest to the latest times. He also published many articles in the technical press, mostly referring to suspension bridges.

As an engineer and architect, Mr. Hildenbrand always showed good taste, and gave close and painstaking attention to all details. His extreme conscientiousness, which made him hesitate to delegate minor parts of his work to subordinates, was the only obstacle to his attaining a still more brilliant success than he achieved.

He had a very interesting and original personality, and was a true friend to many who were fortunate enough to make his acquaintance. He was liberal to a fault in his assistance to many who needed it. He was an enthusiastic admirer of Wagner, played his operas on the flute, and never missed attending their performance when opportunity offered.

He was married, in 1901, to Miss Hubbard, daughter of Judge Hubbard of Covington, Ky., who survives him.

Mr. Hildenbrand was elected a Member of the American Society of Civil Engineers on February 5th, 1902.

FRANKLIN ALLEN HINDS, M. Am. Soc. C. E.*

Died August 23d, 1913.

Franklin Allen Hinds was born on his father's farm near Watertown, N. Y., on November 17th, 1843, and received his elementary education in the public schools of that place. When he was 21, he went, by way of Panama, to Portland, Ore., where he studied under various engineers for two years. He then returned East, and studied for a year at Yale University.

Mr. Hinds then undertook the surveys for the Carthage, Watertown and Sacketts Harbor Railroad, and, later, was made Chief Engineer of that road. Among other railroad work of which he had charge was that for the Kingston and Pembroke, and the surveys for the New York and Boston Inland Railroad. For some years he was in partnership with Mr. John Moffett, constructing municipal water-works in all parts of the United States and Canada.

In 1889, Mr. Hinds formed a partnership with E. A. Bond, M. Am. Soc. C. E., ex-Chairman of the New York Barge Canal Advisory Board. The firm had a large consulting practice in mill construction, water-works, hydro-electric developments, and general engineering. This partnership was dissolved in 1896, but the business was continued by Mr. Hinds until his death. He was City Engineer of Watertown for several years, a member of the Board of Commissioners for 33 years, and a vestryman of Trinity Church for 26 years.

On account of his sterling character and his kindly interest in others, Mr. Hinds was always an inspiring example to the younger men with whom he came in contact. He loved his profession and instilled that love in others. To him, truth and honesty were fundamental, and no design was good, or engineering project sound, unless it was founded on truth and designed with unswerving honesty.

In 1867 he was married to Miss Mary R. Thompson, who, with one brother, Oscar E. Hinds, survives him.

Mr. Hinds was elected a Member of the American Society of Civil Engineers, on May 3d, 1899.

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* Memoir prepared by Lou B. Cleveland, Assoc. M. Am. Soc. C. E.

WILLIAM AUGUST HUNICKE, M. Am. Soc. C. E.*

Died March 9th, 1914.

William August Hunicke was born at St. Louis, Mo., on February 23d, 1875. He attended the public schools until 1889, when he was sent to Germany where he studied for three years. On his return to St. Louis, he entered Washington University, from which he was graduated in 1897, with the degree of B. S. in Civil Engineering.

His first employment was with Robert Moore, Past-President, Am. Soc. C. E., and the late J. B. Johnson, M. Am. Soc. C. E., after which he spent five years with the Mexican Central Railroad, in charge of the location and construction of various branch lines of that Company. Returning to St. Louis, he was placed in charge of the Washington University buildings and the laying out and improvement of the University grounds.

Mr. Hunicke was Assistant Engineer of the St. Louis Terminal Railroad during the time that Mr. Taylor was Chief Engineer, and in that capacity had charge of the relaying of the track in the tunnel, which work had to be accomplished without interference with the traffic. His next work was the location and construction of the extension of the railroads owned by the Sligo Furnace Company in Missouri.

From 1909 to 1911, he was Chief Engineer and Superintendent of Construction of the Apalachicola Northern Railroad, locating and building that road from Apalachicola to Jacksonville, Fla.

From 1911 to 1913, he was Superintendent of Construction for the English Syndicate in Cuba on various extensions of its roads, returning to St. Louis in August, 1913, to become Assistant Operating Engineer of the Missouri Pacific Railroad, which position he held at the time of his death.

Mr. Hunicke was a member of the Engineers' Club of St. Louis, the Railway Club of St. Louis, and the Missouri Athletic Club where he made his home, enjoying its comforts and environments. He was one of those who perished in the fire which destroyed the building on March 9th, 1914.

He was one of the best posted authorities on railroad location, construction, and valuation, and acted as Consulting Engineer on valuation work for a number of the smaller railroads of the South. Beloved by his friends, his was a life well spent in constructive work, and those who knew him well cherish his memory.

Mr. Hunicke was elected an Associate Member of the American Society of Civil Engineers on March 2d, 1904, and a Member on January 7th, 1913.

* Memoir prepared by Leo C. Dziatzko, Esq., Webster Groves, Mo.

NED HERBERT JANVRIN, M. Am. Soc. C. E.*

Died July 17th, 1913.

Ned Herbert Janvrin was born at Somerville, Mass., on May 20th, 1871. He was the son of Hiram Gilmore and Catherine Marriott Plummer Janvrin. An ancestor was one Jean Janvrin of the Isle of Jersey, whose son, Captain John Janvrin, sailed from Lisbon, Portugal, in 1696, and settled in Portsmouth, N. H. John, a son of Captain John Janvrin, born in 1707, was graduated from Harvard College in 1728. William Janvrin, the great-great-grandfather of Ned Herbert Janvrin, married Abigail Adams, a niece of President John Adams. A maternal ancestor, Francis Plummer, emigrated from Wales, at the foot of the Snowden Mountains, in 1635, and with his wife and two sons settled on the bank of the Parker River in Newburyport. He was the first man to keep a tavern and operate a ferry in that part of Massachusetts.

Mr. Janvrin received his early education in the schools of Somerville, having been graduated from the High School in 1890. In the fall of that year he entered the Massachusetts Institute of Technology, from which he was graduated in 1894 with an S. B. degree. George F. Swain, Past-President, Am. Soc. C. E., writes of him that "he was a conscientious, faithful, and very capable student, lending his influence toward a proper standard of discipline and attainment, and he left the school with the respect and confidence of all his teachers."

Almost immediately after his graduation he entered the drafting-room of the Boston Bridge Works, serving there until May, 1895, under J. R. Worcester, M. Am. Soc. C. E., Chief Engineer, who was impressed with his brightness, quickness of comprehension, and accuracy.

From May to November, 1895, he was employed as a Traverseman with the United States Geological Survey on survey work then in progress in New York State. He then returned to the drafting-room of the Boston Bridge Works, remaining there until December, 1896. From that time until May, 1897, he was with Mr. Worcester, during which period the train-shed for the South Station, at Boston, was being designed. Mr. Janvrin did a great deal of responsible work in the way of calculating and checking, particularly in connection with the girder work of the midway floor. In this position he showed that he was developing as an all-around designer, retaining all he had learned by experience and earlier training.

From May, 1897, to July, 1899, he was employed in the Bridge and Construction Department of the Pennsylvania Steel Company, at Steel-

*Memoir prepared by Robert Ridgway, M. Am. Soc. C. E.

ton, Pa., as a Checker and as Engineer in charge of drafting on the designs of steel bridges and viaducts.

Between July, 1899, and March, 1900, Mr. Janvrin was Assistant Bridge Engineer for the Metropolitan Street Railway of Kansas City, Mo., designing several of the Company's bridges and viaducts. He then entered the service of Messrs. Waddell and Hedrick, of that place, remaining with them until August, 1900, during which time he was connected with the design and construction of the Kansas City Viaduct.

From September to December, 1900, he was again with Mr. Worcester, assisting in the design of the steelwork for the Boston Elevated Railway Company, and from January to March, 1901, he served with Norcross Brothers, Worcester, Mass., as a Draftsman on steel building construction.

In April, 1901, he went with the American Bridge Company, remaining with it until September 15th, 1905. Until August, 1901, he was a Draftsman in the Pencoyd Plant. From then until June, 1903, he was Draftsman and Assistant Engineer, respectively, in the Eastern Division drafting-room, then located at Pencoyd, Pa., and in the Eastern Division estimating-room, also at Pencoyd. From June, 1903, to the middle of September, 1905, he was employed as Assistant Engineer in the Erecting Department of the Pittsburgh Division. His work there was mainly in connection with contracts for which the Company had sublet the erection to others. Among such works may be mentioned the erection of the Louisville and Nashville Shops, at South Louisville, Ky., and the Norfolk and Western Bridge, at Portsmouth, Ohio. The notable qualities of his character, which were impressed on his superiors on this work, were his absolute honesty, his quiet, industrious habits, and his ability as a thoroughly competent field checker of detailed drawings.

From October, 1905, to April, 1906, Mr. Janvrin was with Herbert C. Keith, M. Am. Soc. C. E., Consulting Engineer, of New York City, as a designer on several heavy steel bridges for the New York, New Haven and Hartford Railroad. Here he showed a keen perception of the points involved in several special problems which were presented, and great ability in solving them.

On April 2d, 1906, he reported for duty as Assistant Engineer with the Board of Water Supply, City of New York, and remained in its service, engaged on designs and surveys for and the construction of the Catskill Water Supply System, until his death. His purpose in taking up this work was due largely to a desire to increase his knowledge in a branch of engineering different from that in which he had been so long engaged. During this period, as opportunities offered, he received several promotions in rank and pay, on the recommendations of his superiors, for efficient service. Until September, 1906, he

was assigned to the Reservoir Department, on general survey work for the Ashokan Reservoir, during which period he had charge of a stadia party on preliminary topographical surveys. He was then transferred to the Designing Division of Headquarters Department, and in the following spring was placed in charge of the construction and testing of an experimental section of reinforced concrete pipe, 11 ft. in diameter and 210 ft. long. The experiment was made with a view of ascertaining whether this type of construction was feasible and desirable for some of the smaller siphons of the Catskill Aqueduct where the head to be provided for was not great. The pipe consisted of seven 30-ft. units, each unit differing from the others as to the mixture of concrete, the details of reinforcement, etc. Mr. Janvrin gave the details of this work his earnest and intelligent attention, and his report of it, and of the successful hydrostatic test which followed the construction, was of material assistance to the designing force and added much to the knowledge of the subject of reinforced concrete pipes. In December, 1907, he was transferred to the Northern Aqueduct Department and assigned to its Newburgh Division, in Orange and Ulster Counties, where he assisted in locating the cut and cover aqueduct on that Division of 15 miles. When the work was placed under contract, he was at first given charge of Contract No. 16, a section of aqueduct $2\frac{1}{2}$ miles long, and, later, in June, 1909, of Contract No. 45, about 5 miles long. As the intensity of the work increased, Contract No. 45 was divided into two engineering sections, and on April 1st, 1910, Mr. Janvrin was given charge of the south section. A notable feature of his section was the building, in two places, of the large concrete aqueduct of 500 000 000 gal. daily capacity on a heavy foundation embankment of earth, where the surface of the ground fell below the elevation of the aqueduct invert. The larger of the two embankments was 900 ft. long, and its maximum height, from the surface of the ground to the bottom of the aqueduct structure, was about 18 ft. On the practical completion of this section, on May 1st, 1913, he was transferred to the Designing Division of Headquarters Department, where he remained until his death. In this last period of his work, his thorough grounding in the principles of stresses and strains in structures and his complete knowledge of steel and concrete structures were of great assistance in establishing methods of design applicable to the many subsurface gate-chambers of the City Aqueduct tunnel.

While on a vacation he was stricken with spinal meningitis, from the effects of which he died at Boston, Mass., on July 17th, 1913. For nearly 20 years he had given his earnest attention to the active practice of his profession, and his death, in his forty-second year, ended a career of much promise.

A friend of Mr. Janvrin, who had known him from his boyhood, commenting on his characteristics and ideals, writes:

"The one thing that stands out is his single-mindedness. It has been my lot to know of no man whose life from beginning to end was so perfectly straightforward, whose character was more truly clear and clean, whose ideals were more high, but this singleness of mind stands out the most sharply. There was nothing complex in his make-up, and it seems to me in these days of mixed motives in the characters of the best of men oftentimes, that such perfect singleness of mind was as remarkable as it is rare. This same singleness extended to his intellectual processes. His really brilliant mind moved straight ahead always, without waywardness. In all his social relations, the same characteristic predominated. As a boy he was a generous, kindly companion and a helpful son. As a man he was upright and honorable with the world at large, staunch and loyal to his friends, unselfish and devoted toward all his family."

Another, who became associated with him in his work soon after his graduation from college, and who formed a friendship then which lasted as long as he lived, writes of him as

"A man of clean habits and high ambitions, of excellent ability, strictly honest and honorable in all his dealings, and true to his friends."

Those who knew Mr. Janvrin well recognize these traits, and will endorse the tributes to his character paid by these friends.

He was a Mason, a member of Temple Lodge, 299, Kansas City, Mo., a member of the Technology Club of New York, and of the National Geographic Society. He was interested in outdoor sports and skilled in games.

He was married, on June 20th, 1907, to Miss Avis Genevieve Grimes, of Franconia, N. H., who survives him.

Mr. Janvrin was elected a Junior of the American Society of Civil Engineers, on October 5th, 1897; an Associate Member on June 5th, 1901; and a Member on April 4th, 1911.

THOMAS HUMRICKHOUSE JOHNSON, M. Am. Soc. C. E.*

Died April 16th, 1914.

Thomas Humrickhouse Johnson was born at Coshocton, Ohio, on January 12th, 1841. His parents, William Kerr Johnson and Elizabeth (Humrickhouse) Johnson became residents of Coshocton in their youth, during the first quarter of the last century.

William Kerr Johnson was born in Dungannon, County Tyrone, Ireland, of Scotch-Irish parentage, in 1809, and, while yet a boy, came to America with his brothers, residing first in Baltimore and moving West, in 1821, to the Village of Coshocton.

Elizabeth Humrickhouse was a descendant of a Prussian family which emigrated to America from the Duchy of Hesse Darmstadt in the early part of the 18th century, settling first at Bethlehem and Germantown, Pa., and at Hagerstown, Md. From Hagerstown, Peter Humrickhouse, the father of Elizabeth, moved West, finally settling in Coshocton.

Mr. Johnson's early education was under the tutelage of the Rev. William E. Hunt, then pastor of the Coshocton Presbyterian Church, under whom he was fitted for college.

In 1858 he entered old Jefferson College, at Canonsburg, Pa., and after three years was graduated at the age of twenty years, with the degree of Bachelor of Arts, in the class of 1861. On account of his unusual proficiency in mathematics, the degree of Master of Arts was conferred on him in 1866 by his Alma Mater, and in 1911 the degree of Doctor of Science by Washington and Jefferson University, the successor to the former colleges of Washington and Jefferson.

On graduation Mr. Johnson entered the employ of the Pittsburgh and Steubenville Railroad, of which his father was a director, as Rodman; and in 1863 he became Assistant Engineer in the construction of the Steubenville Bridge over the Ohio River. From 1867 to 1869 he served as Assistant Engineer on general work on the Pittsburgh, Cincinnati and Chicago Railway, from 1869 to 1871 on the construction of the Chartiers Railway, and in 1871 was transferred to Richmond, Ind., where he was engaged in the erection of the passenger station at that place until May 31st, 1872. From that date until March 1st, 1873, he served the Company in the general work along its lines, and then began the erection of the depot at Columbus, Ohio. On the completion of this work, in 1875, he was again transferred to Indiana, and through out that year was in charge of construction of the Vincennes Engine House and Richmond Shops.

In the latter part of 1875 he severed his connection with the railroad and opened an office in Columbus, Ohio, for architectural work, and

* Memoir prepared by J. P. Snow, J. C Bland, and R. Montfort, Members, Am. Soc. C. E.

thereafter until March, 1883, was engaged as Chief Engineer in the construction of the present State Capitol at Indianapolis.

In 1883 he again entered the service of the Pittsburgh, Cincinnati, Chicago and St. Louis Railway, as Principal Assistant Engineer, which position he occupied until 1896, when, on the death of M. J. Becker, Past-President, Am. Soc. C. E., he was appointed to the position of Chief Engineer of the Southwest System. In 1901 he became Consulting Engineer for all Lines West of Pittsburgh, a position which he held until January, 1911, when he reached the age (70 years) of automatic retirement.

The services of Mr. Johnson were considered so valuable by the management of the Pennsylvania Railroad that the positions of Chief Engineer of the Pittsburgh, Chartiers and Youghiogheny Railroad, the Chartiers Southern Railway, and Special Consultant for the Pennsylvania Lines were created for him. This procedure, which is without precedent in the history of the Pennsylvania System, constitutes a striking tribute to his capacity and worth. It is a tradition among the Pennsylvania men that no subject was ever broached to him which failed to elicit information to the point and offhand.

In addition to his regular duties, his advice and counsel were frequently sought, and highly valued in cases where expert judgment was required in difficult constructions, and in cases of adjustment between parties with opposing interests.

Mr. Johnson found especial enjoyment in association, not only with his fellow-engineers, but with all men of learning, and to that end he accepted many invitations to membership in scientific bodies. Besides membership in the American Society of Civil Engineers, he was a member of the American Railway Engineering Association; the American Society for the Advancement of Science; the American Society for Testing Materials; the American Forestry Association, and the Engineers Society of Western Pennsylvania. He was a member of the Rail Committee of the American Railway Engineering Association, of the Special Committee on Valuation of Public Utilities, of the American Society of Civil Engineers, and of the Rail Committee of the Pennsylvania System.

Of several of these societies he has at times been on the board of officers, and has served them on committees and as a delegate to various congresses. The Engineers Society of Western Pennsylvania is especially indebted to him, not only as an active officer, but as an adviser and mentor in times of stress, and as one ever ready to carry the burden of unusual activities and grave responsibilities whenever the needs of the society called him.

He was a valued contributor to the publications of many of the associations of which he was a member. To the American Society of Civil Engineers he contributed a notable paper "On the Strength

of Columns; Discussing the Experiments which have been Accumulated, and Proposing New Formulas,"* and discussions on the cause and prevention of decay of building stone; a formula for the strength of columns; Progress report of Special Committee on Steel Columns and Struts; railway bridge designing; stresses in railway bridges on curves; and ventilation of tunnels. His paper and discussion on the Strength of Columns brought out the famous straight-line formula which he demonstrated to be the tangent and rational extension of Euler's curve. His discussion on the ventilation of tunnels was the result of his translation of a work by the Italian Government on tunnel ventilation, an abstrusely mathematical treatise which Mr. Johnson translated with the aid of an Italian dictionary and grammar in combination with his extraordinary mathematical ability.

He was 61 years of age when he translated this work, and when one of his engineer friends remarked on his industry and courage in undertaking such a task, he pointed to a page of the work and asked, "what constitutes the principal part of that page?" The friend answered, "mathematical expressions." "Yes," said Mr. Johnson, "a universal language."

In 1868 Mr. Johnson married Miss Martha E. Patterson, of Steubenville, Ohio, who survives him with two daughters, Misses Bessie and Margaret Johnson, and one son, William K. Johnson, who is an attorney at law in Pittsburgh, Pa.

As a man, Mr. Johnson's notable characteristics were: perfect sincerity, single-mindedness, consideration for others, kindliness, charity, and a constant readiness to help others; and his utter uncomplaining, even in the hour of death. His vast fund of well-ordered knowledge was always at any one's command; and the younger men went to him for advice, counsel, and help as freely as a child to its father.

John Henry Newman says:

"The true gentleman carefully avoids whatever may cause a jar or a jolt in the minds of those with whom he is cast; all clashing of opinion or collision of feeling, all restraint, or suspicion, or gloom, or resentment; his great concern being to make every one at their ease and at home. He has eyes on all his company; he is tender toward the bashful, gentle toward the distant, and merciful toward the absurd; he guards against unreasonable allusions, or topics that may irritate; he is seldom prominent in conversation and never wearisome. He makes light of favors when he does them and seems to be receiving when he is conferring. He never speaks of himself except when compelled; never defends himself by a mere retort; he has no ears for slander or gossip; is scrupulous in imputing motives to those who interfere with him, and interprets everything for the best. He is never mean or little in his disputes, never takes unfair advantage;

* *Transactions*, Am. Soc. C. E., Vol. XV, p. 517.

never makes personalities or sharp sayings for arguments or insinuates evil which he does not say out. * * *

"He is patient, forbearing, and resigned, on philosophical principles; he submits to pain because it is inevitable; to bereavement because it is irreparable, and to death because it is his destiny."

Such a man was Thomas H. Johnson.

Mr. Johnson was elected a Member of the American Society of Civil Engineers on September 5th, 1877, and served as a Director during the years 1900, 1901, and 1902.

FRANCIS VALENTINE TOLDERVY LEE, M. Am. Soc. C. E.*

DIED AUGUST 17TH, 1913.

Francis Valentine Toldervy Lee, son of Francis V. T. Lee, of Shropshire, England, an Officer of the Queen's Own Light Infantry, was born at Winchester, England, on August 28th, 1870. He was educated at the Manchester Grammar School, at Manchester, England, and the College Communal, at Boulogne, France, and, in 1897, was graduated from Leland Stanford, Jr., University, with the degree of A. B. in Electrical Engineering.

In 1887, Mr. Lee went to Sherbrooke, Que., Canada, and for three years was engaged as Private Secretary to the Chief of Construction of the Canadian Pacific Railway. In 1890 he resigned this position in order to supplement with a more adequate technical training part of the education he had received abroad. After a visit to his home in England, he returned to New York City, where, in 1892, he entered the employ of the Manhattan Electric Light Company, as Assistant to the Superintendent, in order to test his liking for the work before specializing in Electrical Engineering.

In 1893 he entered Leland Stanford, Jr., University, and there met the late Dr. F. A. C. Perrine, then Professor of Electrical Engineering, which meeting resulted in one of the great friendships of Mr. Lee's life. As his Secretary and General Laboratory Assistant, he came intimately in contact with Dr. Perrine during his college life, and so strong was his influence that many of Mr. Lee's old friends often remarked on the little personal mannerisms which each had acquired unconsciously from the other.

Shortly after his graduation in 1897, Mr. Lee was appointed Assistant Engineer to Mr. John Martin, District Engineer for the Pacific Coast Department of the Stanley Electric Manufacturing Company. He rose rapidly in the service of this Company, being appointed District Engineer in 1898, Manager in June, 1899, and, in 1900, in addition to his position as Pacific Coast District Manager of the Stanley Electric Manufacturing Company and many other Eastern electrical manufacturers, he was made Vice-President and General Manager of John Martin and Company, Electrical Engineers and Contractors. During this period he had direct supervision of the erection of many of the earlier lighting and power plants which, later, were absorbed by the Bay Counties Power Company and the Pacific Electric Railway Company.

In April, 1906, Mr. Lee severed his connection with John Martin and Company, but followed Mr. Martin's interests into the Pacific

* Memoir prepared by the Secretary from information supplied by A. H. Babcock, Cons. Elec. Engr., Southern Pacific Co., San Francisco, Cal.

Gas and Electric Company, where he served successively as Assistant to the President, in charge of the Engineering and Electrical Departments, Chairman of the Engineering Committee, as well as Assistant General Manager in charge of and responsible for the construction and operation of the hydraulic developments of the Company.

In 1910, Mr. Lee resigned his position with the Pacific Gas and Electric Company, and until a few months before his death spent the time with his family at his old home in England and traveling on the Continent. He had returned to Victoria, B. C., Canada, which city he had intended to make his future home, only a few months before his death.

On September 27th, 1899, he was married to Edith K. Bonnallie, of Sherbrooke, Que., Canada, who, with two daughters, Ruth and Margaret, survives him.

Mr. Lee died before much of his work, particularly that of the last seven years, had time to demonstrate its real worth. In all his business life, his relations with the really big men with whom he worked brought a mutual confidence and personal regard which, in many cases, amounted to real affection. For the others, those of less caliber, he had a good-humored tolerance, although, at times, his path was made exceedingly rough.

His absolute faith in the kindliness of human nature was wonderful, for he had many rebuffs. They never embittered him, however, and he refused to believe any harm or evil of any one until he had absolute proof of it. Many times he was heard to say "'They say,' is a liar," and he lived up to this saying. Those who came intimately in contact with him knew the absolute integrity, the uprightness, and the sweet disposition of the man, and are thankful for their memory of him.

Mr. Lee's personal tastes were simple. The fine arts, of which he had a cultured enjoyment, appealed to him strongly. His reading covered a wide range, and, having leisure, he enjoyed his fine reference library to the full. A list of the works therein is an index of his versatility, and is a revelation even to his intimate friends.

At the time of his death, he was a Member of the American Society of Mechanical Engineers, the Institution of Electrical Engineers, the American Institute of Electrical Engineers, the American Gas Institute, and the American Electrochemical Society. He was also a Member of Occidental Lodge, F. & A. M., of California Chapter, R. A. M., and of Golden Gate Commandery, K. T., all of San Francisco.

Mr. Lee was elected a Member of the American Society of Civil Engineers on February 1st, 1910.

DAVID NEILSON MELVIN, M. Am. Soc. C. E.*

DIED JANUARY 27TH, 1914.

David Neilson Melvin was born in Glasgow, Scotland, on July 21st, 1840. He was the son of David Melvin, of Paisley, Scotland, a successful card manufacturer at Oxford, England, and also a notable figure in the temperance movement in Great Britain.

David Neilson Melvin received his early education at the Andersonian Institute in Glasgow, and, in 1855, was apprenticed to the firm of Crawhall and Campbell, Engineers, to study drafting and to work through the shops.

In 1861, on the completion of his apprenticeship, Mr. Melvin was employed as Draftsman and Mechanical Engineer by several firms in Glasgow and vicinity, particularly by Blake, Barclay and Company, for which company he designed and superintended the construction of machinery and fireproof buildings, for some of the largest sugar-refining mills in Great Britain. He also designed the machinery and buildings for sugar mills in the West Indies.

In July, 1863, he purchased an interest in a paper mill near Oxford, England, which he operated until the abolition of the British tariff on paper made the business unprofitable. In 1865, he went to Birmingham, England, as Assistant to Mr. Henry Lea, Civil Engineer, remaining in that position until April, 1867, when he came to the United States. Shortly after his arrival Mr. Melvin obtained a patent for an improved sectional safety steam boiler, and, later, for an automatic furnace door, and also for a high-pressure engine. He was also associated with Mr. T. A. Weston, the inventor of the differential chain pulley, in Buffalo, N. Y.

Later, he was employed as Engineer and Superintendent by the Rodgers Iron Manufacturing Company, of Muskegon, Mich., and A. F. Bartlett and Company, of East Saginaw, Mich., in the production of wood-working machinery and in the manufacture of steam engines. He also superintended the erection of some of the largest lumber mills in the Michigan lumber regions.

In February, 1873, The American Linoleum Manufacturing Company was organized, and Mr. Melvin was appointed its Engineer. With Mr. Frederick Walton, the inventor of linoleum, he designed and superintended the erection of the machinery and buildings of the Company's large plant at Linoleumville, on Staten Island, New York. On the completion of this work, he succeeded Mr. Walton as Superintendent.

* Memoir prepared by George A. Parker, Esq., Mech. Asst. to Supt., The American Linoleum Mfg. Co., Linoleumville, N. Y.

In 1888, Mr. Melvin invented and patented the machinery for manufacturing an inlaid linoleum, and about 1900, he brought out a patent for wood inlaid. These goods are now being manufactured exclusively under his patents. He retained his position as Superintendent until his death, which occurred at Miami, Fla., on January 27th, 1914, after a lingering illness.

Mr. Melvin was one of the original members, and a Life Member, of the American Society of Mechanical Engineers. He was also a Member of the Richmond County Automobile Society.

David Neilson Melvin was elected a Member of the American Society of Civil Engineers, on July 3d, 1878.

BENJAMIN FRANKLIN MORSE, M. Am. Soc. C. E.*

DIED FEBRUARY 17TH, 1914.

Benjamin Franklin Morse was born on June 7th, 1829, in South Kirtland, Geauga County, now Lake County, Ohio, and in 1836 became a resident of Painesville, in the same county. During his residence at Painesville, he attended the common schools and afterward the Painesville Academy; later, he studied mathematics and civil engineering with General E. A. Paine, a graduate of West Point, who had retired from the United States Army.

Mr. Morse was identified with much of the pioneer railroad building in Northern Ohio, in the capacity of Assistant Engineer, in which connection he represented the Lake Shore Railroad, aiding in the construction of the line between Cleveland and Erie, Pa., in 1851 and 1852.

In 1853, he was Assistant Engineer, under Major Potter, at the harbors of Fairport, Ashtabula, and Conneaut, Ohio. During the season of 1854 he was First Assistant, under Captain Howard Stansbury, U. S. Engineer, in the examination of the harbors on Lake Erie west of Cleveland, namely Lorain, Vermilion, Huron, Sandusky, and Monroe, Mich.

As First Assistant Engineer, Mr. Morse had charge of a line from Tiffin, Ohio, to Fort Wayne, Ind., now a part of the Nickel Plate System. During 1855 he was Assistant Engineer in charge of a survey for The Cleveland and Mahoning Railroad, from Youngstown, Ohio, to New Castle, Pa., and from 1857 until 1862, he was First Assistant, under Mr. Charles Collins, in the Engineering Department of the Lake Shore Railroad, between Cleveland and Erie, Pa.

In 1862 the four railroad companies, the Cleveland and Columbus, the Cleveland, Painesville, and Ashtabula, the Cleveland and Toledo, and the Cleveland and Pittsburg, proposed through their Presidents to build the present Union Station in Cleveland. The Presidents constituted the Building Committee, with Mr. Amasa Stone, of the Lake Shore, as Chairman, and he appointed Mr. Morse as his Engineer. The latter drew the plans for the station, and they were approved by Mr. Stone. They included what was probably at that time one of the largest train-sheds in the United States. Mr. Morse superintended the building of the Union Station, completing the work in 1865.

In 1868, as Chief Engineer, he surveyed the line from Chardon to Youngstown, Ohio, which is now a branch of the Baltimore and Ohio Railroad, extending from Fairport to Youngstown. As Chief Engineer,

* Memoir prepared by J. F. Morse, Esq.

he also made a preliminary survey for a railroad from Cleveland to Sharon, Pa.

Mr. Morse afterward became interested in the erection of many of the public buildings in Cleveland and superintended their construction. He superintended the construction of the City Work House and drew plans and superintended the rebuilding of the Newburg State Hospital, which was destroyed by fire in 1872.

In April, 1875, Mr. Morse was appointed City Engineer of Cleveland, in which capacity he served for 9 years. He remodeled the plans and completed the Superior Street Viaduct, which was opened in 1878. While acting as City Engineer, he estimated and reported on several high-level bridges, and one of the plans he advocated, the Central Viaduct, was adopted and developed.

He also first suggested and planned for the intercepting sewer for the City of Cleveland. He was appointed by the Building Committee to superintend the construction of the Society for Savings Building, but before active work was begun, he engaged with the Lake Shore and Michigan Southern Railroad Company to look after the building and rebuilding of its stations at Toledo and Chicago.

In 1890, under the new Building Code, Mr. Morse was appointed Building Inspector, and served in that capacity for 3½ years. After retiring from this office he spent some time in travel, and laid aside active business except for occasional consultation work in engineering lines.

On September 19th, 1913, Mr. Morse was seriously injured in an automobile accident, from the effects of which he died on February 17th, 1914, at the Battle Creek Sanitarium where he had gone for treatment.

In 1855, Mr. Morse was united in marriage to Matilda Craft, of Tiffin, Ohio. He is survived by three sons and one daughter.

He was a Royal Arch Mason, and for many years belonged to the old Board of Trade, and also to the Chamber of Commerce, of Cleveland.

Mr. Morse was one of the oldest members of the American Society of Civil Engineers, having been elected a Member on July 12th, 1877. He was a Charter Member of the Civil Engineers Club of Cleveland, now the Cleveland Engineering Society.

GEORGE ALFRED NELSON, M. Am. Soc. C. E.*

Died June 3d, 1913.

George Alfred Nelson, the son of George and Abigail Marion Bigelow Nelson, was born in Lincoln, Mass., on September 20th, 1852. He attended the Lexington District schools and the Lincoln High School and was graduated from the latter in 1872. In 1873, he entered the Massachusetts Institute of Technology from which he was graduated in Civil Engineering in 1877. During a summer vacation, from June to September, 1875, he served as Rodman on the survey and construction of the Boston, Concord and Montreal Railroad, and as an Assistant in the office of the City Engineer, at Concord, N. H.

After his graduation from the Massachusetts Institute of Technology, Mr. Nelson spent two years at his home in Lincoln, and was engaged in surveying and various engineering works in that vicinity. In 1879, he went to Lawrence, Mass., where he was employed as sketch maker in the Designing Department of the Pacific Mills. He remained in this position until August, 1883, when he resigned to become Assistant Engineer in the office of the City Engineer at Lowell, Mass., which position he held until his death.

As Assistant Engineer, Mr. Nelson had charge of the design and construction of several bridges in Lowell, one of which was the Taylor Stone Arch Bridge across the Concord River, the location of which involved difficult foundations, and its successful completion showed the thought and skill devoted to its design. He also had charge of a complete survey of the city for assessors' maps; of all water-works improvements; the extension of the sewerage system into new territory; and the design and cost estimates for the abolition of grade crossings within the city limits.

Mr. Nelson was a man of strong character and his personality and keen mind impressed all with whom he came in contact. He had never been very strong physically, and his constant and close attention to the details of his work affected his health so that he was forced to seek rest at frequent intervals. His death occurred on June 3d, 1913, after an illness of a few months. Mr. Nelson had never married. He is survived by a sister and two brothers.

He had always shown much cleverness with his pencil and crayon, and was an expert photographer. He had shown his photographs at various exhibitions in the United States, receiving many medals for his artistic work. With a few others, he was selected to represent the United States at an international exhibition at Berlin, Germany, where his photographs won for him a silver medal.

* Memoir prepared by the Secretary from material on file at the Society House.

Mr. Nelson was an active member of the Eliot Congregational Church Society, at Lowell, and for many years was President of the John Eliot Literary Society, contributing greatly to the success of its work by his active energy and personality. He was a member of the Alumni Association of the Massachusetts Institute of Technology, the Technology Club of the Merrimac Valley, of which he was an active president for several years, and the Association of the Class of 1877 of the Massachusetts Institute of Technology. He was also a member of the Boston Society of Civil Engineers, having designed the pin which was adopted by that Society. He was devoted to outdoor sports and took an active part in the snowshoe trips of the Appalachian Mountain Club, of which he was a member. He was also a member of the Vesper Country Club, at Lowell, and an expert golfer.

George Alfred Nelson was elected a Member of the American Society of Civil Engineers on April 4th, 1911.

PETER ALEXANDER PETERSON, M. Am. Soc. C. E.*

DIED NOVEMBER 21ST, 1913.

Peter Alexander Peterson was born in Niagara Falls, Ont., Canada, on November 8th, 1839. In July, 1859, after a thorough education, he was articled as a pupil in Surveying to Thomas C. Keefer, Past-President, Am. Soc. C. E., who, at that time, was Chief Engineer of the Hamilton Water-Works and of the Hamilton and Port Dover Railway. For two years, Mr. Peterson was engaged on these works, with headquarters at Hamilton, and from 1861 to 1863, on a survey for a canal from Georgian Bay to Toronto, being stationed at the latter place. In July, 1863, he obtained a license as an Ontario Land Surveyor, and during the next two years was employed on various engineering works under Mr. Keefer.

In May, 1865, Mr. Peterson left Mr. Keefer's employ to engage in the private practice of engineering, and during 1865 and 1866, he had charge of the reconstruction of three large dams on the Grand River, to replace those which had been carried away by floods caused by ice jams.

In the summer of 1867 he made surveys, plans, and estimates for the Petrolia Branch of the Great Western Railway, and, in the autumn of the same year, was appointed Resident Engineer of the Northern Division of the New York and Oswego Midland Railway. In March, 1868, he was appointed Resident Engineer of the Bathurst Division of the Intercolonial Railway, in New Brunswick, which was then being constructed.

Mr. Peterson remained with the Intercolonial Railway until September, 1872, when he resigned to accept the position of Chief Engineer of the Toronto Water-Works, the construction of which, it was estimated, would cost $2 000 000, and included a filtering basin 3 000 ft. long, 10 000 ft. of conduit from basin to pumping well (4 500 ft. of which was a 36-in. flexible pipe laid across Toronto Harbor), pumping engines, reservoir, and more than 100 miles of distribution pipes.

In 1875, Mr. Peterson was appointed by the Quebec Government as Chief Engineer of the Montreal and Ottawa Section of the Quebec, Montreal, Ottawa and Occidental Railway, which, except for that portion between Terrebonne and Montreal, had been constructed between Quebec and Ottawa. The location of the unconstructed portion was under discussion and Mr. Peterson strongly urged the adoption of the direct line from a point near Berthier to Montreal *via* Bout de l'Isle, including a large bridge at the latter place. In July, 1878, however, the late Mr. Walter Shanly made a report to the Government of Quebec

* Memoir prepared by H. Irwin, Esq., Cons. Right-of-Way and Lease Agent, Canadian Pacific Railway Company, Montreal, Que., Canada.

in which he advocated the line finally adopted, namely, *via* Terrebonne, St. Vincent de Paul, and St. Martin's Junction, with two large bridges and two long, heavy grades up to and down from Mile End. Mr. Peterson's route was afterward adopted by the Canadian Northern Quebec Railway.

As an engineer, Mr. Peterson excelled in the location and building of the substructures of bridges, and as Chief Engineer of the Montreal and Ottawa Section of the Quebec, Montreal, Ottawa and Occidental Railway, he had charge of the construction of the Chaudière Bridge between Hull and Ottawa, having one span of 254 ft., one of 160 ft., ten of 150 ft., and one of 135 ft., and of several other bridges over the Rouge, North Nation, Lievres, and Gatineau Rivers.

In 1881, he was appointed Chief Engineer of the Atlantic and North West Railway Company, under the charter of which the Canadian Pacific Railway Company built the line from Mile End Station, near Montreal, to connect with the International Railway of Canada at Lennoxville. At the same time, but under another charter, this road was extended from the International Boundary, near Megantic, to Mattawamkeag, Me. While in this position, Mr. Peterson superintended the construction of the St. Lawrence Bridge at Caughnawaga, on this line, the piers of which, with the necessary lengthening, were strong enough to carry the heavy double-track steelwork which, by a strange coincidence, was completed only a month before his death. On this line were also several large steel bridges and trestles, including the bridge over the Richelieu River and high trestles at Ship Pond and Wilson Stream.

Mr. Peterson was also Chief Engineer of the Sault Ste. Marie Bridge and of the Mission River Bridge, in British Columbia, and of many other important works.

In 1890, he was appointed Chief Engineer of the Canadian Pacific Railway Company, which position he held until February, 1902, when he was obliged to resign on account of ill health. He then became the Consulting Engineer of the Company.

In August, 1903, he was appointed Chief Engineer of the Guelph and Goderich Railway then being constructed by the Canadian Pacific Railway Company, and held that position until 1908 when he was obliged to retire from active work owing to his failing health. He made his home in Montreal, Que., Canada, where he died on November 21st, 1913.

He was elected a Member of the Institution of Civil Engineers of Great Britain on December 1st, 1874. He was also a Charter Member of the Canadian Society of Civil Engineers, and served as Vice-President in 1889, 1892, and 1893, and as President in 1894.

Mr. Peterson had taken a leading part in railway construction in Canada during the latter part of the nineteenth century. He was

most conscientious in regard to the performance of his engineering work, and strict, but fair, in his dealings with others. He was very kind-hearted, and always willing to aid any of his staff, although he required from them close attention in their work and whole-hearted discharge of their duty to their employers. Being of rather nervous disposition, he was, at first, sometimes thought to be slightly abrupt in manner, but his unvarying courtesy soon removed that impression.

Peter Alexander Peterson was elected a Member of the American Society of Civil Engineers on January 5th, 1876. He served as a Director in 1892 and 1893, and as a Vice-President in 1896 and 1897.

GEORGE BROWNE POST, M. Am. Soc. C. E.*

DIED NOVEMBER 28TH, 1913.

George Browne Post, the son of Joel B. and Abby M. Post, was born in New York City, on December 15th, 1837. He was educated at Churchill's Military School at Sing Sing, N. Y., and the Scientific School of New York University, from which he was graduated as Civil Engineer with the Class of 1858.

After his graduation, and during 1858, 1859, and 1860, he studied Architecture with the late Richard M. Hunt. In the latter year he formed a partnership with a fellow-student, Mr. Charles D. Gambrill, for the practice of Architecture.

During the Civil War, Mr. Post served in the Army for 4 months in 1862 and for 4 months in 1863, as Captain in the Twenty-second Regiment, New York Volunteers. At the first battle of Fredericksburg, he acted as Volunteer Aide on the staff of General Burnside, commanding the Army of the Potomac. Mr. Post was promoted to the rank of Major, Lieutenant-Colonel, and then Colonel of the Twenty-second Regiment.

After the War he resumed his professional career, and his partnership with Mr. Gambrill was dissolved. In 1905, with his sons, William S. and J. Otis Post, he formed the firm of George B. Post and Sons.

Among the buildings designed by Mr. Post during his long career, are the Prudential Life Insurance and the Mutual Benefit Life Insurance Buildings in Newark, N. J.; the Wisconsin State Capitol; the Cleveland Trust Company Building, in Cleveland, Ohio; and the Manufacturers' Liberal Arts Buildings at the Chicago Exposition in 1893. In New York City, he designed the buildings for the College of the City of New York; the New York Produce Exchange; the New Stock Exchange; the Pulitzer Building, and the Western Union Building, in Dey Street. He also designed the Equitable Life Assurance Society Building, and several well-known private residences.

Mr. Post was a Member of the Architectural League of New York, and served as President from 1893 to 1897, inclusive. He was elected an Honorary Life Member in 1912. He was a Fellow of the American Institute of Architects, of which body he was President from 1896 to 1899, inclusive. He was a member of the New York Chapter of the American Institute of Architects, and President in 1904; of the Fine Arts Federation of New York, and President in 1898. He was a Charter Member of the National Arts Club, and President from 1898 to 1905. He was a member of the Municipal Art Society, and served as Director from 1901 to 1909. He was a member of the Council of

* Memoir prepared by the Secretary from information supplied by Geo. B. Post & Sons.

the National Sculpture Society in 1904. He was also a Member of the National Institute of Arts and Letters, the American Academy of Arts and Letters, the Province of Quebec Association of Architects, New York Academy of Sciences, American Geographical Society, National Society of Craftsmen, Public Art League, Archæological Society of America, National Geographical Society, and the Metropolitan Museum of Art. In 1907, he was appointed Honorary Corresponding Member of the Royal Institute of British Architects. He was elected an Associate of the National Academy of Design in 1907 and an Academician in 1908.

In 1901, Mr. Post was decorated a Chevalier de la Legion d'Honneur of France. In 1908 he received the honorary degree of Doctor of Laws from Columbia University, and in 1910, he was awarded the Gold Medal of the American Institute of Architects.

When the Tenement House Commission, known as the Gilder Commission, was appointed by the New York State Legislature, he was made a member, and, in 1902, he was appointed a member of the Board of Commissions of the St. Louis Exposition by the Governor of New York.

Mr. Post was appointed, by the Secretary of State, as a delegate to represent American Architects at large at the World's Congress of Architects held in London, and, in 1906, the Secretary of Agriculture appointed him a collaborator of the Forest Service of the United States Department of Agriculture. In 1906, President Roosevelt made him a member of the National Advisory Board on Fuels and Structural Materials, to which Board he was reappointed in 1907, 1908, and 1909. In 1909, he was appointed a member of the Bureau of Fine Arts by President Roosevelt.

He was appointed a member of the Committee of Patronage to the Eighth International Congress of Architects in 1907, and in 1908 a Member of the Permanent Committee.

He was a member of the Expert Committee to appoint a sculptor and select a design for the Lafayette Monument erected in the courtyard of the Louvre in Paris.

In 1863, Mr. Post was married to Miss Alice M. Stone, daughter of William W. Stone.

He was a member of the New York Chamber of Commerce, the New Jersey State Chamber of Commerce, Century Association, Union Club, Cosmos Club of Washington, D. C., Lawyers' Club (Charter Member), and the New York Farmers Club.

Mr. Post was elected a Member of the American Society of Civil Engineers on September 2d, 1896.

WILLIAM NAPIER RADENHURST, M. Am. Soc. C. E.*

DIED APRIL 23D, 1913.

William Napier Radenhurst, the son of John and Mary Radenhurst, was born in Toronto, Ont., Canada, on November 5th, 1838. He attended the Rensselaer Polytechnic Institute during 1854, '55, and '56, and pursued studies in Civil Engineering. As a young man, Mr. Radenhurst was employed on the Grand Trunk Railway, and later traveled in New Zealand and New South Wales.

He was appointed Draftsman in the Department of Docks, New York City, by the late Gen. George B. McClellan, and served in that capacity, and as Surveyor and Inspector, until he resigned on January 1st, 1876. He was afterward connected with the Western Division of the New York State Canals, under the State Engineer, Robert Van Buren, M. Am. Soc. C. E., and Division Engineer Evershed. During this period he made surveys of Niagara Falls and vicinity, when the State took over the Park Reservation.

In 1883, Mr. Radenhurst became Assistant Engineer, under the late J. Nelson Tubbs, M. Am. Soc. C. E., Chief Engineer, of the Executive Board of the City of Rochester, N. Y., and retained that office under Emil Kuichling, M. Am. Soc. C. E.

He was appointed Water-Works Assistant Engineer on January 1st, 1900, under the City Engineer, Edwin A. Fisher, M. Am. Soc. C. E., which position he retained for eight years, when he was appointed Assistant Engineer in charge of Permits and Subways. He retired from active professional work on January 1st, 1910.

He was married in 1867 to Frances Hawksley, of Toronto, Ont., Canada, who survives him.

Mr. Radenhurst was a man of artistic tastes and had a wide acquaintance with literature. Although of a retiring disposition, he had many warm friends. He was fond of children, and, having none of his own, he and his wife reared and cared for a number from childhood to maturity.

Mr. Radenhurst was elected a Junior of the American Society of Civil Engineers on July 7th, 1875, and a Member on July 7th, 1880.

* Memoir prepared by John F. Skinner, M. Am. Soc. C. E.

CHARLES WALKER RAYMOND, M. Am. Soc. C. E.*

DIED MAY 3D, 1913.

Charles Walker Raymond was born at Hartford, Conn., on January 14th, 1842, and was the second son of the Reverend Robert R. and Mary A. (Pratt) Raymond. During his childhood his parents removed to Syracuse, N. Y., where he was educated in the public schools and High School until 1857 when, his father becoming Professor of the English Language and Literature in the newly established Collegiate and Polytechnic Institute of Brooklyn, N. Y., the family moved to that city, and he entered that institution. As a child, he had been intensely interested in literature, but backward and awkward in mathematics. In the Polytechnic he came under the influence of Professor Richard Smith, a retired officer of the Engineer Corps (afterward President of Girard College), who was Professor of Mathematics; and, to the surprise of his family, he began to distinguish himself in that study. The strangest feature of this change was that he developed, under the inspiring touch of his brilliant teacher, not a previously unsuspected genius for mathematics, but a high ambition to conquer by persistent work the field which seemed so difficult. It was by hard work at every step that he acquired a thorough knowledge which placed him at the head of his class, and led Professor Smith to urge that he should seek an appointment as cadet in the United States Military Academy, on account of his unusual mathematical ability. In these days, when it is the fashion to educate boys mainly in the lines to which they show the earliest bent, this instance of training in directions not congenial is suggestive. We hear much about the subsequent failures of those who took the highest rank in school; but when that distinction is won, not through the easy superiority of genius, but by intense devotion to distasteful work, the man who wins it is likely to keep it always. This, at least, was the keynote of General Raymond's career, and the secret of his remarkable record of unbroken success through more than forty years of professional service.

Graduated from the Polytechnic in 1860, he was appointed in 1861 to the Military Academy, where he stood at the head of his class for four years. On his graduation, in 1865, he was immediately assigned to the Corps of Engineers with the full rank of First Lieutenant—skipping the rank of Second Lieutenant altogether. In his "furlough year" (1863), he had spent his leave of absence as an officer on the staff of Major-General Couch, by special appointment of the Secretary of War. General Couch commanded the Department of the Susque-

*Memoir prepared by Dr. R. W. Raymond and Alfred Noble, Past-President, Am. Soc. C. E.

hanna, and Lieutenant Raymond's service with him in June and July covered the Campaign and Battle of Gettysburg.

From October 1st, 1865, to September 17th, 1866, he was Assistant to the Special Board of Engineers for improving the fortifications near Boston, Mass. He was then transferred to the Pacific Coast, where he served for a brief period as Assistant Engineer in the construction of the defences of Alcatraz Island, San Francisco Harbor, and then became Recorder of the Board of Engineers for the Pacific Coast. This position he held from December 28th, 1866, to March 3d, 1869, during which period he was promoted (March 1st, 1867) to be Captain in the Corps of Engineers, and also conducted important operations at Lime Point, San Francisco Harbor, where he was Assistant Engineer for two periods—April 24th to July 7th, 1868, and November 5th, 1868, to March 3d, 1869. The work at Lime Point involved the removal of large masses of rock by heavy blasts. The interval between the two periods named was occupied with the reconstruction of Fort Stevens, at the mouth of the Columbia River, Oregon.

In 1869, after the purchase of Alaska by the United States, the American Fur Company, which had succeeded to the business of the Russian Fur Company, sent a small party up the Yukon River as far as Fort Yukon, a post of the Hudson Bay Company, to investigate the fur trade of the upper river and decide whether to establish an American station there. Captain Raymond was ordered by the Commanding General of the Department of the Pacific to accompany this party, for the purpose of making a reconnoissance of the lower Yukon, and determining astronomically the position of the meridian which constitutes the Eastern Boundary of that part of Alaska. Accompanied by one assistant and furnished with an astronomical field outfit, he joined the traders' party, which left San Francisco on April 6th, 1869, in a sailing brig, carrying on deck a small steam pinnace, and bound for Sitka, at which point his party was reinforced by one private soldier detailed for this service. Leaving Sitka on May 9th, the expedition, after many delays, reached the mouth of the Yukon, launched the little steamer, and proceeded in it up the river to Fort Yukon, a distance of, perhaps, 1 000 miles. During this trip, Captain Raymond and his assistant, relieving each other day and night, made a reconnoissance from which he afterward prepared the map of the lower Yukon which accompanied his official report and which was undoubtedly the best which had appeared up to that time. Arriving at Fort Yukon on July 31st, 1869, the traders' party soon ascertained that the fur trade of that region was not large enough to warrant the establishment of an American agency; and, as the freezing of the northern tributaries of the river was already threatening to make it too shallow for their steamer, they resolved to return at once, leaving Captain Raymond and his two companions to finish the astronomical

work and make their way to the coast by themselves, but promising to hold the brig for them at the mouth of the river until a certain date. Establishing a field observatory, and observing the solar eclipse of August 7th, Captain Raymond determined the meridian boundary, proved that the British post was on American territory, and raised the American flag over it. (The post was immediately removed to the other side of the meridian.)

Leaving Fort Yukon on August 28th, his little party descended the river in a rude boat constructed by themselves—the Indians refusing to venture the voyage in their canoes at that season. This vessel conveyed them precariously as far as Anvik (about 500 miles from Fort Yukon and 500 miles from the sea), where it went to pieces. The remainder of the journey was characterized by hardship, exposure, and even starvation; and the explorers, exhausted and emaciated, reached the appointed rendezvous just as the brig was hoisting sail for San Francisco.*

From January 7th, 1870, to June 16th, 1871, he was Secretary of the Board of Engineers for Fortifications, etc., etc., of the United States; and from the latter date to August 23d, 1872, he commanded the Engineer Company at Willets Point, N. Y. On August 28th, 1872, he became Principal Assistant Professor of Natural and Experimental Philosophy at the U. S. Military Academy, and filled this position until February 27th, 1874, and again from August 31st, 1875, to July 1st, 1878—the interval being spent on special service in command of the U. S. Expedition to Northern Tasmania to observe the transit of Venus. From August 28th, 1878, to August 27th, 1881, he was Instructor in Practical Military Engineering, Signaling and Telegraphy, and during a part of that period served also as Superintending Engineer of Construction of the West Point Water-works, the Cadet Hospital, and the Cadet Barracks Extension. In the various positions mentioned, Captain Raymond spent more than seven years in actual service at the Academy.

During this period, he published an essay on Terrestrial Magnetism, which added to his already established reputation as a mathematician and physicist.

From August 30th, 1881, to January 13th, 1883, he commanded the Engineer Company at Willets Point. In January, 1883, he was placed in charge of river and harbor improvements, surveys, and coast defences in Massachusetts, and retained this position until February, 1886. The following condensed catalogue of his duties, compiled from the Annual Reports of the Chief of Engineers, will give some notion of their extent and variety: The protection and improvement of Boston Harbor, in which he established a practicable 28-ft. channel to

* Captain Raymond's official report, published as Senate Executive Document No. 12. of the Forty-second Congress, says little about the sufferings of his party, but they can be read between the lines. His health was permanently impaired.

the ocean; operations of similar character at the Harbors of Newburyport, Scituate, Plymouth, Provincetown, Lynn, etc., and Sandy Bay; studies and improvements of the Merrimac and Malden Rivers, and numerous other preliminary examinations, surveys, and projects. The most important of these works, from an engineering standpoint, was the improvement of Newburyport Harbor, at the mouth of the Merrimac, where Captain Raymond, through modifications of plan and method, secured, not only an increase of channel depth on the bar from 7 to 18.5 ft., but a saving in expense over the original scheme, which contemplated only a 17-ft. channel. His report on Sandy Bar (1884) contains a brilliant and novel discussion of the cross-section of a breakwater, and the anchorage capacity of a harbor, which contributed new ideas to engineering literature. On February 20th, 1883, he was promoted to be Major in the Corps of Engineers.

During his term at Boston, he served also for the greater part of 1883 and 1885 as Engineer of the First and Second Lighthouse Districts (covering the Coast of Massachusetts and Maine); and from October, 1883, to April, 1884, he superintended the removal of a wreck from Gloucester Harbor.

In February, 1886, he was ordered to take charge of levees and other improvements on the Mississippi River, from Warrenton to its delta, and from that position he was called, December 7th, 1886, to Washington, D. C., as Assistant to the Chief of Engineers. This office (which often made him temporarily Acting Chief of the Corps), he retained until January 26th, 1888, when he was appointed Engineer Commissioner of the District of Columbia—one of the three Commissioners constituting the government of the District. He served until February 1st, 1890, dealing, among other works of municipal engineering, with the difficult problem of electrical subways, concerning which his official report furnished a valuable theoretical and practical discussion.

From February 13th, 1890, to September 30th, 1902, he was (with the exception of two months in the latter year) continuously in charge of the defenses, harbor improvements, etc., at Philadelphia and in Delaware River and Bay. This work included the completion, on a novel plan and by a method designed by Major Raymond, and with great saving in both time and cost, of the now famous Delaware Breakwater which, disproving all the sinister prophecies of its early critics, stands unmoved after ten years of practical trial, a monument to his courage and originality as an engineer.*

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* This work is described in the "Final Report of Lt.-Col. Charles W. Raymond, Corps of Engineers, Upon the Improvement of Delaware Breakwater," Reports of the Chief of Engineers, U. S. Army, 1899, Part II, p. 1346 *ff*. It is a significant circumstance that when Major Raymond took charge of it, in 1891, a plan had been already approved, in 1890, by the U. S. Board of Engineers, which he was expected to carry out: so that, in order to put his own conception into effect, he had to move that Board to recede from its own recently adopted scheme and adopt a new one, comprising features not supported by engineering precedents.

The following sketch of his work in the Philadelphia District is compiled from the Annual Reports of the Chief of Engineers, from 1890 to 1902, inclusive.

For the first eight years of his tour of duty, the project for the Delaware River up to Philadelphia provided for a depth of 26 ft. at mean low water, and from Philadelphia up to Trenton for a depth of 12 ft. In the upper river above Philadelphia the principal obstruction to the 12-ft. channel was Kinkora Bar. At this point a dike had been built before he took charge. The natural depth was 7.5 ft. He succeeded in obtaining greater depths, but the work done was not permanent in effect.

During these eight years he made annual examinations of the shoals in the river at Smith's Island Bar, Mifflin Bar, Schooner Ledge, Five Mile Bar, Bulkhead Shoal, Cherry Island Flats, Dan Baker Shoal, and other places. At most of the shoals and bars he obtained the projected depth of 26 ft.

Five Mile Bar is just above Philadelphia. It was improved by a dike and by dredging. Smith's Island was completely removed by dredging. Petty Island was partly removed. Ledges of rock were removed by blasting and dredging. Mifflin Bar was improved by dike construction and dredging. Schooner Ledge was deepened by rock removal. Cherry Island Flats was deepened by dredging. Bulkhead Shoal was deepened by the construction of Finn's Point dike. Dan Baker Shoal was originally planned to be deepened by a long dike from Reedy Island to Liston's Point, but this plan was later abandoned by reason of a decrease in the cost of dredging, and the channel was deepened by dredging alone.

In 1898 the condition of the river was good except at a few points. At Dan Baker Shoal the least depth was 16.5 ft. at low tide. In that year, a survey was made for the purpose of determining the possibility of still greater improvement, and on the basis of this survey a Board of Officers recommended a project for a channel from Philadelphia to Delaware Bay, 30 ft. deep and 600 ft. wide. Major Raymond started this improvement, deepening the channel to 30 ft. as rapidly as funds were made available. As the Delaware River is a silt-bearing stream which tends to obliterate artificial channels not placed exactly where Nature would place them, the improvement involves continual maintenance work. Even ten years after he had left the district, the work was still in progress, and it will always be continued as long as the river is used for commerce.

One of the great problems involved in this undertaking consisted in finding suitable places to dispose of dredged material. Major Raymond combined a dike for the improvement of Dan Baker Shoal with a plan for a disposal basin, making in mid-stream an artificial island about three miles long. The bulkhead was built, and the basin was

so large that it has received dredged material for many years and is not yet full. This island is locally known as Raymond Island.*

In May, 1898, he became Lieutenant-Colonel, Corps of Engineers, and in May, 1901, a member of the U. S. Board of Engineers, of which, as a young Captain, he had been the Secretary, 31 years before. He remained a member of this Board until his retirement from active service in 1904.

During this period he completed the great work of an analytical index of the Reports of the Chief of Engineers from 1866 to 1900, inclusive. This book, in three volumes, is a model of intelligent and convenient classification, as well as comprehensiveness and accuracy.

It was in these later years of his life that some of his most important work was performed. As already observed, he served as member on many Boards and Commissions, reporting on fortifications, rivers, harbors of refuge, etc. Among these, the most noteworthy were: The Board to decide between San Pedro and Santa Monica Bay, for the location of a deep-water harbor on the coast of California; the Board to determine the maximum span for a suspension bridge, and especially for such a bridge over the Hudson River at New York; and the

*The following minute of the Executive Council of the Philadelphia Board of Trade shows the estimate placed on his work by that body:

"To the President and Members of the Executive Council of the Board of Trade:

"GENTLEMEN:

"Your Committee on the Improvement of the Harbor and Delaware and Schuylkill Rivers respectfully reports:

"That it has noted with regret the death of General C. W. Raymond, U. S. A., and in view of the services rendered by him to the city of Philadelphia during the twelve years he was in charge of this district as Engineer Officer, presents for your adoption the following minute:

"The Executive Council of the Philadelphia Board of Trade records this minute upon the death of General C. W. Raymond, U. S. A., which took place after a lingering illness May 3, 1913.

"The city of Philadelphia owes a debt of gratitude to General Raymond for his intelligent and untiring efforts for the improvement of the navigation of the Delaware and Schuylkill Rivers and the removal of Smith's and Windmill Islands, which made possible a widened Delaware Avenue, and the building of wharves of such dimensions as to accommodate modern vessels.

"The work on the improvement of the harbor was prosecuted under his direction from the time the title to the islands was vested in the United States, under condemnation proceedings, May 29, 1890, until the completion of the readjustment of the harbor conditions, January 10, 1898, and during all that time he was responsive to every suggestion of the commercial and maritime interests looking to the early and successful termination of the work which has inured so greatly to the advantage of the city and port.

"It was under General Raymond (then Major) that the initial dredging for a 30-ft. channel, as provided by Act of Congress, March 3, 1899, took place, and was continued until he was relieved from duty at Philadelphia on July 20, 1901. He designed and practically carried to completion the great constructions belonging to the National Harbor of Refuge at the entrance of Delaware Bay.

"General Raymond's pre-eminent qualifications as an engineer were universally acknowledged, and his appointment as Chairman of the Commission to plan and construct the tunnel approaches to the New York Terminal of the Pennsylvania R. R. Co., was a practical recognition of his high standing in his profession; and the success of the undertaking furnished additional proof of his unrivaled technical skill and executive ability.

"The members of the Executive Council in adopting this minute, desire to express their high appreciation of the services of incalculable value rendered the port of Philadelphia during the twelve years General Raymond had charge of this district, and at the same time to tender his family their sympathy in the loss sustained by them.

"On motion, this minute was adopted by a rising vote and the Secretary instructed to send a certified copy to the family of General Raymond."

Board to determine the route and cost of a deep waterway from the Great Lakes to the Atlantic.

Of the two latter, he was the President. The report of the Bridge Board contained an analytical discussion of the Theory of Suspension Bridges, mainly prepared by him, a translation of which was subsequently used in Europe as a textbook of instruction. The Deep Waterways Board made careful surveys for a ship canal from Lake Erie to deep water on the Hudson River, below Albany, and, with the aid of maps furnished by the Engineer Corps of the Army for the channels above Lake Erie, prepared detailed estimates for the whole line from the head of lake navigation. The report of the Board, which was the joint work of its members and assistants, was universally recognized as the most complete and thorough of its kind in the literature of engineering.

In January, 1904, he became Colonel, Corps of Engineers; and on June 11th of the same year, he was retired, at his own request, with the rank of Brigadier-General, U. S. A., after more than forty years of consecutive active service. The Government, however, still required him as one of its representatives in the International Congress of Internal Waterways, of which he was, from 1902 until his death in 1913, a member of the Council, Chairman of the American Section, an attendant at the meetings of the Congress, and a contributor to its Proceedings. The latest of these meetings was held in 1912 in the United States, and General Raymond would have been its presiding officer, had he been able to be present.

One of the most memorable labors of his life was the last. Already several years before his retirement, he had been permitted by the War Department, at the urgent request of the Pennsylvania Railroad Company, to act as Chairman of the Board of Engineers created by that Company to supervise the design and construction of the vast improvement contemplated by it in and around New York City, including the tunnels under the Hudson River, the East River, and the Borough of Manhattan; the great Pennsylvania Terminal in New York; and the terminals and yards on Long Island and in New Jersey. General Raymond was not an idle member of that supreme body; all important features of the plans and specifications were passed on by the Board, to the work of which he gave unremitting attention for more than eight years. The many studies and investigations carried out by him during that period were fruitful in determining the final design. Among the most important results of his special labors was the discovery of a minute diurnal rise and fall of the tunnel tube, under the influence of the tides. His recommendation that the tunnel should be left free to move, without any attachment of piles to resist either rise or fall, was adopted by the management of the Company, and has been vindicated thus far by experience. The great tube now ad-

justs itself freely to the changes in pressure of the material surrounding it, and its minute but practically irresistible movements do not affect its stability, in which, by reason of its immense weight, the effect of passing trains is likewise a negligible factor. The name of General Raymond worthily stands at the head of the list of engineers, on the great memorial tablet at the portal of the Pennsylvania Railroad Station in Seventh Avenue, New York City.

These labors were performed under difficulties which might well have discouraged a less intrepid spirit. General Raymond had lost almost entirely, some thirty years before, possibly as the result of the hardships of his explorations in Alaska, the use of one eye; and, about 1900, the formation of a cataract in the remaining eye threatened him with entire blindness. Under this increasing disability (which finally reached such a point that he had to be personally led to and through the tunnels, and could examine drawings only with the aid of a strong glass, magnifying one spot at a time), he continued the active and efficient discharge of his duty as head of the directing Board of Engineers until, in 1910, its work was practically done, and his office in New York was closed. Even after that, in his seaside cottage near Atlantic Highlands, N. J., he dictated, at the request of the Pennsylvania Railroad Company, and under great difficulties of ever-growing blindness, a report of the work of his Board, which has been published by the Company and constitutes a memorable contribution to the literature of that branch of engineering.

Meanwhile, many successive operations (each partly, but not decisively, successful) had been performed on his eye, in which the obscuring film repeatedly gathered. In October, 1912, after months of total blindness, he went to Washington once more for another such operation, with more than usual expectation of permanent relief. The surgeons at Washington, however, discovered another and previously unsuspected trouble—an internal malignant tumor which forbade the expected operation and was itself beyond cure. There was nothing left on earth to him but hopeless darkness and cruel pain, which he bore with characteristic fortitude and patience until his death on May 3d, 1913. Conscious and serene to almost the last moment, he crowned with a heroic death a long, active, useful, and distinguished life, graduating at the end, as he had graduated at the Military Academy nearly half a century before, "at the head of his class." The Alumni of West Point may well be proud of his stainless and illustrious record.

Charles Walker Raymond was elected a Member of the American Society of Civil Engineers on June 1st, 1892.

WALLACE BERKLEY RIEGNER, M. Am. Soc. C. E.*

Died January 19th, 1914.

Wallace Berkley Riegner, the son of Aaron H. and Caroline S. Riegner, was born in Strawsburg, Franklin County, Pa., on January 27th, 1854. In his early boyhood his parents moved to Chambersburg, Pa., where he attended the public schools, from which, as well as from the Chambersburg Academy, he was graduated. He afterward entered Lafayette College from which he was graduated with high honors in June, 1877. While at Lafayette he received the Junior Mathematical Prize, the Senior Astronomical Prize, and was chosen to deliver the Honorary Philosophical Oration. He also represented his College in the Intercollegiate Mathematical Contest.

After his graduation, Mr. Riegner taught mechanical drawing in the public schools of Reading, Pa., for about one year. On October 27th, 1879, he entered the Engineering Department of the Schuylkill Canal where he remained until March 1st, 1880, when he went to Pottstown, Pa., as Assistant Engineer on the Philadelphia and Reading Railroad, remaining in this position until August 31st of the same year.

He then went South and from December, 1880, to December, 1881, was engaged on surveys and construction work, on the Elizabeth City and Norfolk Railroad. He afterward returned to the Engineering Department of the Schuylkill Canal, with headquarters at Reading, Pa.

In April, 1882, Mr. Riegner resigned his position with the Schuylkill Canal Company to enter the employ of the Philadelphia and Reading Railway Company. From April, 1882, to July, 1883, he was engaged as Division Engineer on the Shamokin, Sunbury and Lewisburg Division, and from July, 1883, to April, 1887, as Assistant Engineer on general field and office work and the design of railroad structures at the company's office in Philadelphia, Pa. In April, 1887, he was appointed Engineer of Bridges, which position he held at the time of his death which took place at Chambersburg, Pa., on January 19th, 1914.

Mr. Riegner was a Member of the American Society for Testing Materials, the Franklin Institute, and the Engineers' Club of Philadelphia.

He was an intelligent, energetic, and capable engineer, modest to a degree, with a strong grasp of details which enabled him to prosecute his work most successfully.

Mr. Riegner was elected a Member of the American Society of Civil Engineers on September 7th, 1904.

* Memoir prepared by William Hunter, M. Am. Soc. C. E.

BAIRD SNYDER, Jr., M. Am. Soc. C. E.*

Died July 9th, 1913.

Baird Snyder, Jr., was born at Pottsville, Pa., on November 21st, 1868. He was the third son of Baird and Edith Morris Snyder. The mother, now deceased, was a great-granddaughter of Robert Morris, so distinguished in the days of the American Revolution and a signer of the Declaration of Independence. His father, Baird Snyder, was the son of George Washington Snyder, one of the early pioneers of the Anthracite Region, and the three generations have been prominent in the development and progress of mining in the district.

On his graduation from the Pottsville High School in 1885, Mr. Snyder entered the service of the Pennsylvania Railroad Company as Clerk in the office of the Chief Engineer.

In 1888 he joined the Engineer Corps of the Philadelphia and Reading Coal and Iron Company, and began the engineering career of which his friends and associates are so justly proud and which broadened in experience and grew in accomplishment until his valued life was suddenly closed.

In 1893, the late Joseph S. Harris, while President of the Lehigh Coal and Navigation Company, recognizing Mr. Snyder's ability, appointed him Assistant Superintendent of the mining operations of that Company at Lansford, Pa. Three years later he was made General Manager of the Company, and remained in this position until January, 1912.

The property which came under his management is one of the oldest and best in the Anthracite Region. The various adverse conditions which arise in the experience of the anthracite mine manager were all present during Mr. Snyder's administration at Lansford. The many difficulties of mining, pumping, ventilation, labor, fighting fire under ground, and countless others, were met with patience and self-reliance and were successfully overcome. Among Mr. Snyder's most marked qualifications for his position was his ability to handle men. No man had more loyal subordinates to aid him, and no men ever had a more intrepid leader.

After nineteen years of active and aggressive work at Lansford, he resigned from his position with substantial expressions of confidence and regard from his Board of Directors and with the heartfelt regret of his subordinates.

On leaving the Lehigh Coal and Navigation Company, Mr. Snyder, as President and General Manager, organized the Locust Mountain Coal Company, for the purpose of operating an undeveloped area

* Memoir prepared by Mr. Frank A. Hill, Gen. Mgr., Maderia, Hill & Co., Inc., Pottsville, Pa.

of coal land owned by the Stephen Girard Estate. He went into this enterprise with his characteristic fearlessness and vigor. He had completed his financial and working plans, and was in the midst of their early development with every confidence of success, when death came.

He had met and conquered the many engineering difficulties that had come to him in his old work, and was looking forward with pleasure to solving the new and less complicated questions which would meet him in opening an undeveloped territory of his own selection, but on July 9th, 1913, from injuries received in an automobile accident on the preceding day at Wapwallopen, Pa., this strong and virile life, filled with the promise of many more successful years, was cut off "in the twinkling of an eye", leaving to his family and his many friends and professional associates only the memory of an active and well spent life, full of achievement; the record of an honest man, a good citizen, an able engineer—a record creditable to the membership of any engineering society.

Mr. Snyder was a man of splendid physique, of fine mentality, a reader, and a student. Positive, even to brusqueness, without fear, and a natural leader, he was also a devoted son, a loving husband, and a kind father. He is survived by his widow, Jennie Craig Romig Snyder, and two sons, Baird Snyder, 3d, and Robert Morris Snyder.

Mr. Snyder was elected a Member of the American Society of Civil Engineers on March 2d, 1904.

NATHANIEL TURNER, M. Am. Soc. C. E.*

Died January 19th, 1914.

Nathaniel Turner was born in Scituate, Mass., on March 27th, 1861, and was educated at the public schools of that place, and at Bingham Academy.

Leaving home at the age of nineteen, Mr. Turner went West and began work as a Rodman on the location of the Milwaukee, Lake Shore and Western Railroad, in Wisconsin, in 1880. In 1881, he was made Transitman on the same work.

In 1882, Mr. Turner left the employ of the Milwaukee, Lake Shore and Western Railroad and went to Mexico where he was appointed Levelman on the line of the Mexican National Railway then being constructed between Laredo and the City of Mexico. He remained with this Company until 1888, having served as Transitman, Resident Engineer, and Bridge Engineer, respectively. In 1884, he was employed on trigonometrical surveys of large land areas in the State of Coahuila, Mexico.

In 1888, Mr. Turner was appointed Bridge Engineer and in 1889 Locating Engineer on the Monterey and Mexican Gulf Railway, then building between Monterey and Tampico, under Mr. William H. Wentworth, Chief Engineer. In 1890, he was made Principal Assistant Engineer in charge of construction, remaining in that position until 1892, when he resigned to become Chief Engineer of the Matehuala Porvenir Railway. He also served as Locating Engineer on the Gulf and Pacific Railway. In 1893, he was appointed Chief Engineer of the Monterey and Mexican Gulf Railway, in which position he remained until 1897.

From 1897 to 1901, Mr. Turner was engaged in the private practice of engineering. In the latter year he was appointed Superintendent of Construction of the Monterey Steel Works, a $10 000 000 plant. On the completion of this work in 1904, he again returned to private practice for a year, at which time he went to Matehuala, San Luis Potosi, as General Superintendent of the Matehuala Smelter and Water-Works.

On account of ill-health, Mr. Turner resigned this position in 1908, and retired from active business. He owned a ranch in Tamaulipas and oil investments near Tampico, and devoted his time to these interests, making his home in Monterey. He died of heart

*Memoir prepared by the Secretary from information on file at the Society House.

disease on January 19th, 1914, and was buried in the Carmen Cemetery, at Monterey.

In 1885, Mr. Turner was married to Elizabeth Luella Donaldson, of Rock Island, Ill., who, with one daughter, survives' him. He was one of the best known and best loved Americans in Northern Mexico, and had served as President of the American Colony in Monterey.

Mr. Turner was elected a Member of the American Society of Civil Engineers on March 6th, 1895.

LUTHER REESE ZOLLINGER, M. Am. Soc. C. E.*

DIED OCTOBER 21ST, 1913.

Luther Reese Zollinger, the fifth son of William George and Susannah (Spece) Zollinger, was born on April 25th, 1865, in Harrisburg, Pa. He received his early education in the public schools of his native city, having been graduated from the High School in 1883. Of his early life his most intimate boyhood acquaintance, Dr. C. R. Phillips, of Harrisburg, writes:

"No member of our class in High School, where we were associated for four years, held his classmates by the mere strength of a lovable personality as did Luther Reese Zollinger. We were all devoted to him because of that indescribable something he had, which, unfortunately, is uncommon, and made us certain of his sincerity of purpose and life. He was honor man in our class in High School, easily so too, and I have always felt that he was honor man in our class at Lehigh, though it is true that one or two others had a few per cent. higher standing, according to the poor, insufficient ratings which, as things educational stand, must still be taken to measure a man's mind and ability. Of all the men whom I have known, he had, I always felt, the best mind for mathematical work. In saying this I, of course, except a few neurotics whose mathematical ability always is a part of an unstable mental equilibrium."

In 1884, Mr. Zollinger entered Lehigh University, and was graduated in 1888 with the degree of Civil Engineer. He was President of his class in the Sophomore year, Editor of the College *Epitome,* and Business Manager and Editor of the *Engineering Journal,* and had a Commencement appointment. During his 4 years in college he never ranked lower than fifth, and for the last 3 years he ranked second in a class varying from 101 to 66 men.

Mr. Zollinger's entire professional career was spent in the service of the Pennsylvania Railroad Company, in which he rose by his own efforts and merit from the humble post of Chainman to the position of trust and responsibility which he occupied at the time of his death. Starting in the office of the Assistant Engineer, Middle Division, at Harrisburg, Pa., on March 11th, 1889, he received his first professional experience at the time of the great floods throughout Pennsylvania, in May and June of that year, which proved so destructive to the railroads and other internal improvements in various parts of the State, especially in the vicinity of Johnstown, and along the Susquehanna, Juniata, and Conemaugh Rivers.

He held successively, and for varying periods from 1889 to 1905, the various positions of Transitman in the office of the Engineer of Maintenance of Way, Pennsylvania Railroad Division, at Altoona;

* Memoir prepared by J. F. Murray and C. J. Parker, Members, Am. Soc. C. E.

Assistant to Assistant Engineer, Philadelphia Division, at West Philadelphia; Assistant Supervisor, Division No. 8, Middle Division, Pennsylvania Railroad, at Spruce Creek; Supervisor, Division No. 28, at Norristown; Supervisor, Division No. 1, Philadelphia Division, at West Philadelphia; Assistant to Principal Assistant Engineer, and Principal Assistant Engineer, Pennsylvania Railroad Division, at Altoona.

On April 1st, 1905, Mr. Zollinger was promoted to the position of Engineer of Maintenance of Way, General Office, Philadelphia, which position he held at the time of his death. In this capacity he was in charge and had direct control of the Maintenance of Way Department, in so far as was necessary to insure the efficiency of the Department and adherence to the standards of the Company. He also had charge of the preparation of all maintenance of way plans, the issuing of instructions in regard to adherence to the same, and personally examined all bridges and other structures, reporting on their condition and making recommendations in connection therewith, as well as with respect to other matters relating to maintenance of way.

In addition to these duties, Mr. Zollinger, in 1905-07, was in charge of the construction of extensive yards and yard facilities at Morrisville on the New York Division, and at Shire Oaks on the Monongahela Division; he was also appointed as Chairman of various committees, by the General Manager, to make experiments, investigations, and reports on sundry matters relative to the economical construction and maintenance of track. One of these reports,* dated March, 1911, covers extended experiments to determine the necessary depth of stone ballast. Of this report, W. C. Cushing, M. Am. Soc. C. E., Chief Engineer, Maintenance of Way, Pennsylvania Lines West, says: "These tests are the most extensive of the kind ever conducted in this country, and will be found of great interest and value to railway engineers."

Mr. Zollinger traveled extensively, both in this country and in Europe, and his wonderful powers of observation, retentive memory, quick comprehension, enthusiasm, and capacity for work, made him invaluable to the company he served. As an example of his enthusiasm and wonderful resourcefulness, he purchased a tract of land at Merion, Pa., on which was an old, abandoned stable, and with the aid of his architect transformed it into a beautiful home of the English manor type,† utilizing for the interior finish and decoration the materials from an old mansion formerly on the estate.

His personality was peculiarly lovable and attractive, and left its impress deep on all who met him. One felt immediately that he was a man worth knowing, an impression that association only

* *Proceedings*, American Railway Association, Vol. 13.

† Described by the architect in *House and Garden*, December, 1912.

strengthened and confirmed. Big of body, handsome of feature, with great magnetism, his winning personality drew all classes of men to him, and he held them with hooks of steel by his genial, broad-minded, and tolerant disposition. He was by nature a man of essentially social tendencies. Of fine intellectual gifts, broad education, wide acquaintance with the best literature, discriminating taste in the fine arts, varied experience with many classes of men, an observant traveler, with a retentive memory, great fund of anecdote and reminiscence, and a keen sense of humor, he was always entertaining in conversation. These qualities, with his loyalty, kindness, generosity, and buoyance of spirits, made him a most interesting and delightful companion.

Full of sympathy, he was never too busy to give time to the troubles of his friends or counsel to the youth seeking opportunity. It is not often that a man who has attained high professional standing shows a live interest in the young man just beginning his life work, but Mr. Zollinger was the champion of youth, with an unbounded faith in its capabilities and the fulfilment of its aspirations. This faith he exemplified in a very striking and practical way, for he took more than one young man into his home, gave them his friendship and confidence, inspired them by his manliness, educated them at his own expense, and launched them on honorable and useful careers in life. It was a great satisfaction and source of much pride to him to know that each of these young men fully justified the trust placed in him.

His genial nature, overflowing with good fellowship, so attractive to his many friends, was carried into his domestic circle and was one of the finest phases of his life, as seen by his intimate friends. He never married, and his devotion to his sister can only be described as beautiful. Nothing he did for her was ever felt to be a sacrifice. His happiness was only reached through her happiness. Such affection was warmly reciprocated, and his home was made to him a place of charm as well as of rest.

Mr. Zollinger's dominant characteristic was courageous and vigorous manhood. His power of mind and body was shown in his capacity for concentration and the accomplishment of work. He was of quick discernment and swift comprehension, and his strength made him positive in his convictions and strenuous in maintaining them. With his strong character and self-respect, he could never play the courtier, and he detested to receive the flattery he would never apply to others, not even in the form of studied deference which, at times, might have gained him personal advantage.

Enthusiasm in regard to whatever matter he took up, either work or recreation, was another feature of his character, and made him a natural leader. He was a lover of the beautiful in Nature and art. Nature in all her forms appealed strongly to him. Some of his happiest

hours were spent in roaming among the trees of the forest or the flowers of his own garden, whose characteristics and history he knew so well. He kept in touch with the advancement in science and all questions of the day, and could enjoy intelligently and appreciate intercourse with men versed in many forms of learning.

Men of great professional attainments and mental power are often deficient in those qualities of the heart which make them loved as well as honored, but these qualities Mr. Zollinger possessed in an unusual degree. He had intellectual superiority, and he was a man of honorable achievement, but those who knew him well think of him and love him for his broad-gauged, liberal-minded, vigorous manhood and spotless integrity. He was chivalric in thought and deed, and the courteous gentleman always. Honor, truth, and duty were the stars by which he steered his course, and in him was embodied the highest type of manhood. Of splendid physique, clean and wholesome morals, a fascinating personality with a highly cultivated mind, he died in the prime of brilliant achievements. The exquisite charm of his friendship can be known only to those who enjoyed it; to them it was a benediction, and his death an irreparable loss. In a generation we shall not see his like again.

Mr. Zollinger had complained of ill health for several months prior to his death, though he did not relinquish any of his work, and had just returned from the annual track inspection when he was stricken with apoplexy, on the morning of October 16th, and died at his home in Merion, Pa., on October 21st, 1913.

He was a Member of the American Railway Engineering Association and the Delta Upsilon Fraternity.

Mr. Zollinger was elected a Member of the American Society of Civil Engineers on March 6th, 1901.

GEORGE JOHN COUCHOT, Assoc. M. Am. Soc. C. E.*

Died May 3d, 1914.

George John Couchot was born in Paris, France, on August 23d, 1874. In October, 1887, he came to the United States, going to Oakland, Cal., where he attended the High School.

After a course in an Engineering School, Mr. Couchot, in October, 1892, was engaged by the City Engineer of San Luis Obispo, Cal., on transit work. In March, 1893, he went to the Fulton Iron Works, in San Francisco, as Mechanical Draftsman, remaining in that position until August, 1894. From April to December, 1895, he was employed by the State Harbor Commissioners of California, leaving to go with the Western Sugar Refining Company, with which Company he remained until December, 1896.

From January to April, 1897, Mr. Couchot was employed as Draftsman in the office of Mr. J. C. H. Stut, Mechanical Engineer, and was engaged on the design and construction of the steel frame building for the Western Beet Sugar Factory, at Watsonville, Cal., until August of the same year. He then entered the service of the Fulton Iron Works as Mechanical Draftsman, which position he held until August, 1898.

From October, 1898, to March, 1900, Mr. Couchot was employed as Mechanical and Ship-Building Draftsman by the Union Iron Works, of San Francisco, but resigned to accept again a position with the Fulton Iron Works. In March, 1901, he was appointed Assistant Engineer of the Western Sugar Refining Company, in charge of the design and construction of all that Company's improvements, buildings, machinery, etc.

In February, 1905, he was employed, on the design of steel structures, by Maurice C. Couchot, M. Am. Soc. C. E., Consulting Engineer, with whom he remained until June, when he was appointed Draftsman in the office of the City Engineer of San Francisco. Mr. Couchot retained this position until September, 1907, when he became connected with the firm of Couchot and Thurston, Consulting Engineers, on the design of structures. In August, 1908, he returned to the City Engineer's office where he remained as Chief Draftsman and Assistant Engineer in charge of some of the sewer design and construction, until his death on May 3d, 1914, after an operation for appendicitis.

Mr. Couchot was held in high esteem by all who came in contact with him. His wife and a young son survive him.

Mr. Couchot was elected an Associate Member of the American Society of Civil Engineers, on May 3d, 1910.

* Memoir prepared by the Secretary from material on file at the Society House, supplemented by information supplied by Maurice C. Couchot, M. Am. Soc. C. E.

PHILIP CHAPIN DAVIS, Assoc. M. Am. Soc. C. E.*

DIED MARCH 26TH, 1914.

Philip Chapin Davis was born on September 14th, 1881, at Kalamazoo, Mich., in which city he obtained his early education. In 1906, he was graduated from the Engineering Department of the University of Michigan, with the degree of Bachelor of Science in Mechanical Engineering. In September of that year, he entered the employ of the Thompson-Starrett Company, General Builders, and was with that company on several notable engineering problems, both in Chicago and New York, until August, 1911, when he became associated with the Jobson-Gifford Company of New York, as Superintendent of Construction in connection with the electrification of the New York, New Haven and Hartford Railroad, between New York City and New Rochelle, and also between Stamford and New Haven.

On October 29th, 1908, Mr. Davis was married to Miss Bertha Shean, of Kalamazoo, Mich., who, with two sons, survives him. Mr. Davis died at the Neurological Institute, New York City, on March 26th, 1914, following a year's illness from recurring attacks of pernicious anæmia.

Mr. Davis was a member of Anchor Lodge, No. 87, F. and A. M., and a member of the Sigma Chi Fraternity. He was a young man of exceptional ability, and his friends and associates feel deeply their loss on account of his untimely death.

Mr. Davis was elected an Associate Member of the American Society of Civil Engineers on June 4th, 1913.

* Memoir prepared by Erle K. Knight, Assoc. M. Am. Soc. C. E.

ALBERTO DE LA TORRE, Assoc. M. Am. Soc. C. E.*

Died October 5th, 1912.

Alberto de la Torre, the son of Demetrio de la Torre and Maria Josefa Umaña, was born in Tocaima, Colombia, on August 6th, 1876. He received his elementary education in his native country, and in June, 1889, was sent to the United States to begin the study of Civil Engineering at Rensselaer Polytechnic Institute. He was graduated in 1897, with the degree of Civil Engineer, and returned to Colombia to devote himself to the practice of his profession.

Mr. de la Torre was engaged in private practice in Bogota from March, 1897, to August, 1898, his work consisting of surveying and general engineering. In September, 1898, he was appointed Assistant Engineer of Maintenance of Way on the Ferrocarril de la Sabana, but resigned this position in December, 1898, and engaged again in surveying and general engineering practice.

In April, 1899, Mr. de la Torre accepted the position of Assistant Engineer on the location of the Girardot Railway (now the Colombian National Railway), and remained in this position until the completion of the location to San Joaquin, in July, 1899, when he was appointed Assistant Engineer, under Mr. T. B. Nowell, on the location of the Dorada Railway Extension to the Port of Cambao.

From October, 1899, to April, 1901, he was again engaged in the private practice of surveying and architectural work, and from April to October, 1901, was employed temporarily as Engineer of Way Works on the Dorada Railway.

He was employed as Assistant Engineer on the location and construction of the Girardot Railway from October, 1901, to May, 1904, and, during this time, also had charge of the construction of several private residences, and made the surveys for a cart road on a private estate.

From May, 1904, to July, 1905, Mr. de la Torre was engaged as Engineer and partner of the contractor, on the construction of 2 km. of the Girardot Railway, and from July to December, 1905, he was employed as Assistant Engineer on surveys and estimates for the reconstruction of the Cambao Road. He was afterward appointed by the Government of Colombia to direct the work on the Central Road, between the Puente del Común and Chocontá. In 1908, he again entered the employ of the Girardot Railway Company, being engaged on the construction of the last sections between Zipacón and Facatativá. He was then appointed Resident Engineer in charge of the last section from Girardot to Juntas de Apulo, which position he held until July 1st, 1912.

*Memoir prepared by Mr. Diodoro Sánchez, President of the Colombian Society of Engineers, Bogota, Colombia, supplemented by information on file at the Society House.

Mr. de la Torre was a member of the Colombian Society of Engineers, and the following is a translation of part of an obituary notice which appeared in the November, 1912, issue of the *Anales de Ingenieria,* the official organ of that Society:

"His constant labor for a dozen years, and his continual battle with the inclemencies of our tropical climate, destroyed his energy and weakened his physique to the extent of bereaving his country and his friends of a noble young life which had already rendered distinguished professional services."

Mr. de la Torre was elected an Associate Member of the American Society of Civil Engineers on October 3d, 1906.

MURRAY FORBES, Assoc. M. Am. Soc. C. E.*

DIED DECEMBER 28TH, 1913.

Murray Forbes was born in Philadelphia, Pa., on June 23d, 1863. His father, Dr. William S. Forbes, was Professor of Surgery at Jefferson Medical College. His mother Celinere (Sims) Forbes, was a sister of J. C. Sims, Secretary of the Pennsylvania Railroad Company, and Judge Clifford Sims of the Superior Court of New Jersey.

Mr. Forbes received his early education at Rugby Academy and at Dr. Farries' School, in Philadelphia, and entered the Art School of the University of Pennsylvania at the age of fourteen. When he was 17, he entered the Pennsylvania Railroad shops at Altoona, Pa., where he took the regular 4 years' apprenticeship course. He was then employed in the shops for some months, being afterward transferred to Derry, Pa., where for 4 years he was Assistant Road Foreman of Engines on the Pittsburgh Division of the Pennsylvania Railroad.

He resigned this position in 1888 and went to Greensburg, Pa., where he took charge of the construction of the plant of the Westmoreland Water Company. Mr. Forbes continued as the executive head of this Corporation until his death. Under his direction, the plant was extended and, with allied corporations, served the communities along the Main Line of the Pennsylvania Railroad west from Greensburg as far as Irwin, and south along the Southwest Branch as far as Youngwood. A total of 50 000 people are supplied, in addition to large industrial and mining plants, by the companies in which he was the guiding spirit, and this, too, in a section where, because of the broken topography and the dumping of mine drainage into the streams, the operating conditions are exceedingly difficult and expensive. Mr. Forbes was in responsible charge of the design and construction of the plant, and, at the time of his death, he was, and had been for many years, Manager, Secretary, and Treasurer of the Company and its allied interests.

Mr. Forbes' success at Greensburg resulted in his being chosen to take charge of the construction and operation of water plants for the Derry Water Company at Derry, Pa., and the Dennison Water Supply Company at Dennison, Ohio, serving as Manager, Secretary, and Treasurer of both these corporations. He also had a large financial interest in all three companies.

Mr. Forbes was recognized as an authority in matters pertaining to water-works construction, operation, and valuation. He was employed in a number of water-works cases, either as an expert witness

* Memoir prepared by W. C. Hawley and G. W. Hutchinson, Members, Am. Soc. C. E.

or as a member of boards of arbitration. He assisted in organizing the Pennsylvania Water Works Association and served for 3 years as its President. He was also a Member of the American Water Works Association, the New England Water Works Association, the Engineers' Society of Western Pennsylvania, the Engineers' Society of Pennsylvania, and the Union League Club of Philadelphia. He was a Mason and a member of the Protestant Episcopal Church.

Mr. Forbes was a delightful character, always kind and courteous, a man who made friends readily and who kept them. Of strong convictions, he was also a diplomat. Although for many years he had acted as Manager of the Water Company which served the town in which he lived—and under difficult conditions and with necessarily high rates—he was popular and highly respected in that community. An editorial in a Greensburg newspaper, at the time of his death, reads, in part, as follows:

"Mr. Forbes was in many ways superior. He was a factor in the affairs of his town and of his county. He was intensely human. He was a master of his profession, and he controlled because he knew. A useful man has been taken from this community."

In 1893, Mr. Forbes was married to Miss Ethel Parvin, of Philadelphia, who, with five children, survives him.

Mr. Forbes was elected an Associate Member of the American Society of Civil Engineers, on June 5th, 1907.

ROGER TIFFT HOLLOWAY, Assoc. M. Am. Soc. C. E.*

Died March 12th, 1914.

Roger Tifft Holloway, the second son of Henry F. and Metta J. Holloway, was born on August 29th, 1885, in Columbus, Ohio. In 1887, the family moved to Montclair, N. J., and he was educated at the public schools of that place. He was graduated from the High School in 1904, and was President of his class during his Junior and Senior years. In the fall of that year, he entered Cornell University, from which he was graduated in 1908, with the degree of Civil Engineer. While at Cornell, Mr. Holloway was elected a member of the Alpha Delta Phi and of the Sphinx Head, a Senior society, and he also belonged to the Cornell Glee Club.

In August, 1908, Mr. Holloway was engaged by the Turner Construction Company, of New York City, as Assistant to the Superintendent on reinforced concrete work, remaining with the Company until January, 1909. From March to October, 1909, he was employed as Structural Detailer and Draftsman with the Hay Foundry and Iron Works, at Newark, N. J. In February, 1910, with the writer, he engaged in private practice, as a Consulting Structural Engineer, under the firm name of Mead and Holloway, continuing as a member of the firm for two years. During this time he prepared plans, estimates, etc., for numerous alterations and contracts, among which were plans and specifications for the steelwork for the Irvington School, the Nurses' Home in connection with the Presbyterian Hospital, in Chicago, Ill., the Dock Street Pier, Philadelphia, Pa., etc. He also acted in an advisory capacity on this work.

In June, 1912, Mr. Holloway severed his connection with the firm of Mead and Holloway to engage in the general practice of civil engineering. He was also the New York representative of the London firm of Bagley, Mills and Company, and until his sudden death which occurred at his home in Montclair on March 12th, 1914, from septic poisoning, following an attack of tonsilitis, he was engaged in preparing plans, specifications, estimates, etc., for a number of constructions and alterations in and around New York City.

Mr. Holloway was always courteous, and, in his short professional career, had established a reputation for painstaking care and accuracy which won him many friends. The aptitude shown by him in his profession was probably inherited, his father being a member of the American Society of Mechanical Engineers, and a great-uncle, Mr. J. F. Holloway, having served a term as President of that Society.

* Memoir prepared by Charles A. Mead, M. Am. Soc. C. E., supplemented by information on file at the Society House.

He was a member of the University Glee Club of New York, the Harlequins, of which he was President, the Montclair Athletic Club, the Alpha Delta Phi Club of New York, and Montclair Lodge, No. 144, F. and A. M.

Mr. Holloway was prominent in the social activities of Montclair, where his loss will be keenly felt. His engagement had recently been announced, and the wedding had been planned for the coming Autumn. Besides his parents, he is survived by two brothers and two sisters.

Mr. Holloway was elected a Junior of the American Society of Civil Engineers on May 31st, 1910, and an Associate Member on May 7th, 1913.

GEORGE WILLIAM LEE, Assoc. M. Am. Soc. C. E.*

Died January 6th, 1911.

George William Lee was born on June 10th, 1875, at New Haven, Conn. He was the son of George W. and Harriet (Chappel) Lee, of East Lynn, Conn., where for five generations their ancestors had lived. He was educated at the Bordentown Military Institute, Worcester Academy, and Worcester Polytechnic Institute.

Mr. Lee's whole life was devoted to his Profession, even during his school vacations, for in the summer of 1894 he was engaged on surveys near Winchester, Conn.; in 1895 on surveys for the improvement of the water supply of Athol, Mass.; and in 1896 on surveys for a sewerage system for Hyde Park, Mass., and a water supply for Billerica, Mass.

In 1897 he finished his course at the Polytechnic, and was then engaged by William Barclay Parsons, M. Am. Soc. C. E., and placed in charge of a party making surveys for the Subway in New York City, after which he was with a corps of engineers, U. S. A., on surveys for military roads in Porto Rico.

In August, 1901, Mr. Lee was engaged as Engineer for Sundstrom and Stratton, General Contractors, and remained with this firm until his death, at which time he was Chief Engineer. He showed marked ability in designing plant and directing the construction of many structures. While thus engaged the most important work under his supervision was on the New York Central and Hudson River Railroad, in constructing the Oak Grove and DeWitt Yards, double-tracking the Fall Brook Division and the Peekskill Tunnel on the main line, the Chateaugay Branch of the Delaware and Hudson Company, and Contract No. 3 of the New York State Barge Canal, at Fort Miller, N. Y. His devotion to his work won the admiration of all with whom he came in contact.

He is survived by his wife, Rhoda Hoyt Lee, and two daughters, Harriet E. and Dorothy C. Lee.

Mr. Lee was elected a Junior of the American Society of Civil Engineers on March 4th, 1902, and an Associate Member on January 2d, 1907.

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* Memoir prepared by William R. Hill, M. Am. Soc. C. E.

HORACE GUY MERRICK, Assoc. M. Am. Soc. C. E.*

Died October 30th, 1913.

Horace Guy Merrick was born at Libertyville, Ill., on January 29th, 1879. His family moved to Manistee, Mich., when he was quite young.

Mr. Merrick's education began in the public schools at Manistee, and he was graduated from the High School of that place in 1898. During the next three years he was engaged on various public works in Michigan. In 1901 he entered the University of Michigan, from which institution he was graduated in 1905, with the degree of B. S. in Civil Engineering.

After his graduation he was employed by the United States Government as Junior Engineer on the survey of the Great Lakes until May, 1907, when he was transferred to the work of improving the Upper Mississippi River, with headquarters at La Crosse, Wis. He remained on this work until his death.

In 1913, Mr. Merrick took the examination before the United States Civil Service Board for registration and promotion to Assistant Engineer, and passed with much credit.

He was an energetic and capable civil engineer, and was held in high esteem by his superior officers. He felt no wish to become conspicuous, professionally or otherwise, shrinking from notice rather than courting it. He always under-estimated his own ability, and worked zealously, intelligently, and successfully from love of his Profession.

Mr. Merrick was elected an Associate Member of the American Society of Civil Engineers on May 7th, 1913. He was also a member of the Wisconsin Society of Civil Engineers.

* Memoir prepared by W. A. Thompson, M. Am. Soc. C. E.

JAMES DYNAN NEWTON, Assoc. Am. Soc. C. E.*

DIED AUGUST 8TH, 1912.

James Dynan Newton was born at Oswego, N. Y., on April 17th, 1871. He entered Holy Cross College, Worcester, Mass., in 1887, and was graduated in 1891 with the degree of Bachelor of Arts. He taught sciences in the Roman Catholic High School, Philadelphia, for one year and then entered Sibley College, Cornell University, and received the degree of Mechanical Engineer in 1895. In the same year he was granted a Master's Degree from Holy Cross College.

After his graduation from Cornell he served as Special Apprentice in the New York Central Railroad shops at Oswego for five months. In December, 1895, after passing the examinations, he was appointed Cadet Engineer in the United States Revenue Cutter Service, and in 1896 he was promoted to Third Lieutenant of Engineers. He served in this capacity till 1902, when he was retired for disability incurred in the line of duty. He was in the Revenue Cutter Service during the Spanish-American War, and for a part of this time was Acting Chief Engineer of the U. S. S. *Hamilton*.

He was sent to the Marine Hospital at Fort Stanton, N. Mex., for treatment, and after leaving there was employed on mining development in New Mexico, practiced engineering independently, and, in 1904, was employed as Chainman and Rodman on the Atchison, Topeka, and Santa Fé Railroad. In August, 1905, he accepted the position of Assistant Professor of Civil Engineering in the University of Kansas.

The writer was in charge of the Department of Mechanics during the time of Professor Newton's service in the University of Kansas, and, as a part of his teaching was in this Department, it gave good opportunity for observing his work.

In the Engineering Profession, no doubt, one of the greatest virtues is that of hard, conscientious labor, and this Professor Newton possessed in a marked degree. Entering a new field, he at once saw the necessity of careful preparation. He not only prepared himself for the work in hand, but spent many hours in the study of related subjects. His broad college training and his practical experience enabled him to present his subjects in an interesting and practical manner, and the writer always found his students enthusiastic in his praise.

At the end of his fourth year at the University of Kansas, he was elected Dean of the School of Engineering of Loyola University, Chicago, Ill. Here, his great capacity for labor stood him in hand.

* Memoir prepared by Herbert A. Rice, Assoc. M. Am. Soc. C. E.

The Engineering School, which had not been established, had to be organized, courses laid out, catalogue prepared, equipment purchased for a new building then in process of construction, and instructors engaged. This entire work fell to Professor Newton. Success attended his efforts, and, at the time of his death, the school was thoroughly organized and had enrolled many students in the new Department.

Professor Newton was an enthusiastic worker in the Rcman Catholic Church, being one of the leading soloists in the church choir while in Lawrence.

He was married in 1906 to Miss Minnie Medaris, of Kansas City, who, with one child, survives him. They reside at Lawrence, Kans.

Professor Newton was elected an Associate of the American Society of Civil Engineers on September 5th, 1911.

JOHANNES CORNELIS VLIEGENTHART, Assoc. M. Am. Soc. C. E.*

Died October 29th, 1913.

Johannes Cornelis Vliegenthart was born in Delft, Holland, on July 15th, 1876. In 1893, he entered the Polytechnical School of Delft as a student of engineering, and was graduated therefrom in July, 1899, with the degree of Civil Engineer.

In September, 1899, he entered the Government service of The Netherlands as Assistant Engineer in the Royal Corps of Waterstaat, in charge of river improvement works of the waterway from the Port of Rotterdam to the sea, at Hook of Holland and Hansweert. This work included dredging, constructing groins for regulating the channel, etc.

In November, 1901, Mr. Vliegenthart was appointed Chief Engineer of the Haiho River Conservancy Commission, with headquarters at Tientsin, China. This Commission was appointed by the Peace Protocol after the Boxer Insurrection of 1900, for the purpose of dredging and regulating the Haiho River, with a view to making it navigable from Tientsin to the sea for steamers drawing from 10 to 14 ft. of water. Under his direction three channels were constructed of a total length of 4 miles, thereby shortening the river by 15 miles and eliminating ten sharp bends. In addition to this work, Mr. Vliegenthart had charge of a survey of part of the Province of Chili, which included the cities of Peking, Tientsin, and Paoting. This survey was made chiefly for the purpose of studying the courses of the various tributaries of the Haiho River, looking to the improvement of such tributaries. He had promised to read a paper before the Koninklijk Instituut van Ingenieurs (Royal Institution of Netherland Engineers) on the great engineering works in North China in which he had had such a prominent part, but the fulfillment of this promise was prevented by his death.

With the exception of a vacation in 1906, he remained in China until early in 1913, when he resigned his position with the Haiho Conservancy Commission and returned to his home in Holland. In September, 1913, on the election of the well-known engineer, Mr. R. R. L. de Muralt as a Member of Parliament, Mr. Vliegenthart was appointed to succeed him as Engineer of the "Waterschap Schouwen" (Hydraulic Company, Isle of Schouwen, Province of Zeeland). He remained in this position until his death which occurred on October 29th, 1913, at Zierikzee, Holland.

He belonged to that group of Netherlanders who have raised the reputation of Dutch engineers in foreign countries to a high standard, and his name will be held in honored remembrance as one of the pioneers who co-operated in the development of China.

Mr. Vliegenthart was elected an Associate Member of the American Society of Civil Engineers on June 5th, 1907.

* Memoir prepared by the Secretary from information on file at the Society House.

HENRY HELM CLAYTON, Jun. Am. Soc. C. E.*

Died April 8th, 1913.

Henry Helm Clayton, the son of Lillie Sale and John Benjamin Clayton, was born in Sappington, Mo., on February 8th, 1885. He received his education at the Kirkwood Public School and the Manual Training School at St. Louis, Mo., and was graduated from the latter in January, 1903.

Mr. Clayton began work as Rodman with the Terminal Railroad Association of St. Louis, where he remained until June, 1903, when he accepted a position on the International Mexican Railroad. Having won a scholarship in the Engineering Department of Washington University, he returned to the United States in September, 1903, to enter that University, and in June, 1907, he was graduated with the degree of B. S. in Civil Engineering.

He then went to Texas as Rodman on construction work for the Rock Island Railroad, but during the same year he accepted a position as Computer for the Board of Examination and Survey of the Mississippi River Commission. He remained in this position only a short time, returning to Texas to enter again the employ of the Rock Island Railroad Company, which position he retained until the work there was completed.

In January, 1908, Mr. Clayton formed a partnership for the private practice of engineering, with offices at Clayton and Kirkwood, Mo., which partnership was dissolved in December of the same year. In the meantime he had been appointed City Engineer of Wellston, Mo., but resigned this position to enter the service of the United Railways Company of St. Louis.

In May, 1909, Mr. Clayton joined the forces of the Missouri Pacific Railroad and continued in the employ of that company in various positions until his death on April 8th, 1913. In January, 1910, he was appointed Assistant Division Engineer on the Arkansas Division of the St. Louis, Iron Mountain and Southern Railroad. On June 1st, 1911, he was sent to Chester, Ill., as Assistant Division Engineer on the Illinois Division, where he remained until December 1st, 1912, when he returned to the Southern District.

Mr. Clayton was a man of irreproachable character and of strong personal magnetism, and was unusually energetic in the faithful execution of his work. Having a warm heart and a generous disposition, he was true and faithful as a friend. His make-up was that of a big man, and it is believed that he would have accomplished big things had his life been spared.

Mr. Clayton was a Thirty-second Degree Mason. He was elected a Junior of the American Society of Civil Engineers on December 3d, 1907.

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* Memoir prepared by John B. Clayton, Jr., Esq.

ORLOFF LAKE, Jun. Am. Soc. C. E.*

DIED OCTOBER 21ST, 1913.

Orloff Lake, the eldest son of Duff G. and Mary Ida (Woods) Lake, was born at Baltimore, Md., on April 19th, 1883.

He was graduated from Tulane University with the degree of Bachelor of Engineering in 1905, and immediately thereafter entered the service of the Philadelphia and Reading Railway as Rodman, which position he held until April, 1906, when he resigned to accept the position of Transitman with the New York Central and Hudson River Railroad. He was soon promoted to Assistant Engineer, and had charge of office and field work on construction at Brewster; he also made surveys for the relocation of the New Jersey Shore Line from Weehawken to Shadyside.

In January, 1908, Mr. Lake left the New York Central and joined the engineering forces of the writer as Assistant Engineer on general work in and near New Orleans. He remained in this employ until the summer of 1909 when he temporarily withdrew from the engineering field and transferred his activities for two years to the manufacturing business conducted by his father and uncle.

His love for his profession, however, led him to accept employment, in June, 1911, with the Barrett Manufacturing Company, and he was especially engaged in the Tarvia Department to co-operate with county and other engineers in the South who were devoting their time and talents to highway construction. Mr. Lake's education and training especially qualified him for this work, in which he continued with marked success until his sudden and untimely death.

Mr. Lake was a young man of exceptional attainments and brilliant promise. He was a tireless student and a most industrious worker, whose physical strength was without doubt often overtaxed by the tasks which he set for himself to perform. His exceeding modesty, amounting almost to diffidence, no doubt operated in retarding his professional progress during his brief life. His high ideals, sterling good qualities, and quiet forcefulness, however, were such that, as acquaintance with him ripened, confidence in and affection for him grew strong. In his death, the Profession and the Society have lost a most promising member; and in the zone of his acquaintance there is a void which may not easily be filled.

Mr. Lake was elected a Junior of the American Society of Civil Engineers on December 1st, 1908.

* Memoir prepared by J. F. Coleman, M. Am. Soc. C. E.

HENRY A. RICHMOND, F. Am. Soc. C. E.*

DIED MAY 10TH, 1913.

Henry A. Richmond was born in Syracuse, N. Y., on August 3d, 1840. He was the second son of Dean Richmond who, at the time of his death in 1866, was President of the New York Central Railroad Company.

Mr. Richmond was engaged in the grain business, as a commission merchant, at Buffalo, N. Y., and was well known among shipping circles on the Great Lakes. Later, he was connected with a lithographing business.

After his retirement from active business, he became prominent in public affairs in Buffalo, as an "old-line Democrat". He was President of the Buffalo School Association, and was appointed by the late Grover Cleveland, then Governor of New York, as a Member of the first Civil Service Commission of New York State.

Mr. Richmond never married. He was a "wide traveler and a devotee of art and literature", and was liked and respected by all who knew him. He died at Los Angeles, Cal., on May 10th, 1913, and was buried in the Richmond Mausoleum at Batavia, N. Y.

Mr. Richmond was elected a Fellow of the American Society of Civil Engineers on July 7th, 1870.

*Memoir prepared by the Secretary, from information supplied by C. M. Morse, M. Am. Soc. C. E.

TRANSACTIONS

OF THE

American Society of Civil Engineers

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